

Criteria**Loads and Loading****Contents**

	Page
4.0 Loads and Loading	4.1-1
4.1 Loads	1
4.1.1 Dead Loads	1
4.1.2 Live Loads	1
A. General	1
B. Distribution to Superstructure	1
C. Distribution to Substructure	3
4.1.3 Wind Loads	3
4.1.4 Wind on Live Load	3
4.1.5 Earthquake Loads	3
4.1.6 Other Loads	4
A. Thermal, Shrinkage, and Prestressing	4
B. Buoyancy	4
C. Centrifugal	4
D. Force from Stream Current, Floating Ice, and Drift	4
E. Collision Force on Columns	4
F. Collision Force on Traffic Barrier	4
4.2 Load Combinations	4.2-1
4.2.1 Combination of Loads	1
4.2.2 Load Factor Coefficients	1
4.2.3 Service Load Coefficients	3
4.2.4 AASHTO LRFD Load Factors	4
4.3 Application of Loads	4.3-1
4.3.1 Dead Loads	1
4.3.2 Live Loads	1
4.3.3 Wind Loads	4
4.3.4 Earthquake Loads	4
4.4 Foundation Modeling	4.4-1
4.4.1 Procedure Summary	1
4.4.2 Spread Footings	1
4.4.3 Pile Foundations	1
A. Lateral Spring Input from P-Y Curves	1
B. Lateral Spring Input to Dynamic Analysis	4
C. Vertical Springs	7
D. Stiffness Matrix	8
E. GPILE Computer Program	8
4.99 Bibliography	4.99-1
Appendix A	
4.4-A1-1 Foundation Design Seismic Flow Chart	
4.4-A2 Peak Ground Acceleration Map	
Appendix B	
4.3-B1 Basic Truck Loading	
4.3-B2 Common Response Modification Factors	
4.3-B3 Seismic Analysis Example	
4.4-B1 Spring Constants Evaluation Example	

4.0 Loads and Loading

AASHTO Standard or AASHTO LRFD loading specifications shall be the minimum design criteria used for all bridges.

4.1 Loads**4.1.1 Dead Loads**

Use values in AASHTO except as herein modified:

Reinforced Concrete — 160 pounds per cubic foot.

D.L. Forms in Top Slab of Concrete Box Girders — 5 pounds per square foot of cell area.

4.1.2 Live Loads**A. General**

Live load design criteria is specified in the lower right corner of the bridge preliminary plan sheet. The Bridge Projects Unit determines this criteria using the following guideline:

- HL 93 — New bridges designed in accordance to AASHTO LRFD Specifications.
- HS 25 — New bridges designed in accordance to AASHTO Standard Specifications.
- HS 20 — Detour bridges and bridge widenings with no addition of substructure.

Use values described in AASHTO. Design for HS25 loading by multiplying HS20-44 axle loads by 1.25. The loading consisting of two 24K axles at 4-foot centers sometimes governs for short span bridges. See Figure 4.3.2-1 for illustration of this “alternative” loading.

See Figures 4.3.2-2 and 3 for “L” value to use in the formula in Section 4.3.2. Figure 4.3.2-2 illustrates determination of the “L” length of the member under consideration. For beams and girders, use span length center to center of supports. For cantilevers, use length from center of support to farthest load on cantilever. See Figure 4.3.2-2 for illustration.

B. Distribution to Superstructure**1. Integral Deck Precast Sections**

The Live Load Distribution factor for Bulb Tee, Single Tee, and Double Tee bridges shall be as determined in the AASHTO Specifications.

The AASHTO Specifications should be used for Rib Deck Bridges and the beam types listed therein. For Rib Deck Bridges use a K value of 2.2.

Examples of beam types are shown on Figure 4.1.2-1.

2. Concrete Box Girders

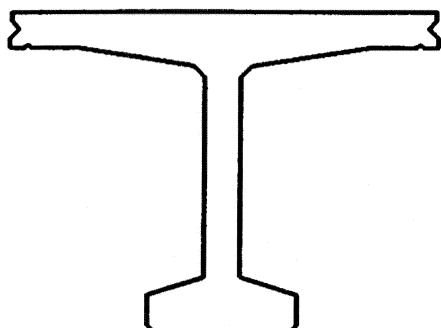
The value for the number of traffic lanes to be used in the concrete box girder superstructure design shall be determined by dividing the entire roadway slab width by 14. Use fractional lanes, rounding to the nearest tenth of a foot, if applicable. Roadway slab widths of less than 28 feet shall have two design lanes. No reduction factor will be applied to the superstructure for multiple loadings.

BRIDGE DESIGN MANUAL

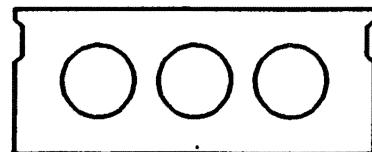
Criteria

Loads and Loading

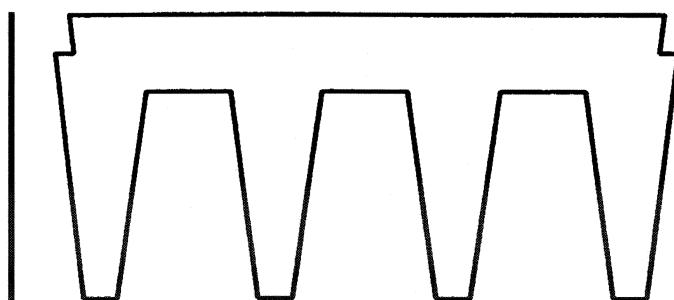
Loads



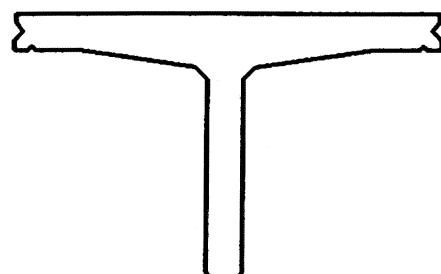
BULB TEE



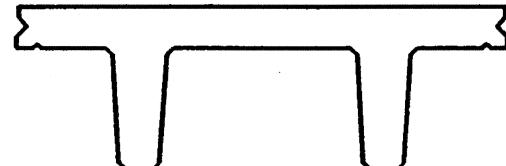
RECTANGULAR
BEAM WITH
CIRCULAR VOIDS



RIB DECK



SINGLE TEE



DOUBLE TEE

Beam Types
Figure 4.1.2-1

Criteria**Loads and Loading****Loads**

3. Other Types

Per AASHTO Specifications.

C. Distribution to Substructure

The value for the number of traffic lanes to be used in the substructure design shall be determined by dividing the entire roadway slab width by 12. No fractional lanes shall be used. Roadway slab widths of less than 24 feet shall have a maximum of two design lanes. A reduction factor will be applied in the substructure design for multiple loadings in accordance with AASHTO. The following percentages of the resulting live loading shall be used:

<u>Number of Lanes Loaded</u>	<u>Percent</u>
Two Lanes	100
Three Lanes	90
Four Lanes or More	75

4.1.3 Wind Loads

Design for wind shall be in accordance with AASHTO LRFD Specifications Standard Specification, load combinations for wind are based on probability of simultaneous load occurrence. The basic wind loads result from 100 mph wind, which produces 75 psf on trusses and arches, 50 psf on girders and beams, and 40 psf on substructures. This wind is assumed to act on the structure when live load is not present. A 30 mph wind (0.3 × 100, or a 70 percent reduction from basic) is included in Groups III and IV, and is assumed to act when live load is present.

The forces tending to overturn a structure are represented by an upward high wind pressure of 20 psf acting on the plan view area, for Groups II, V, and IX. A moderate wind pressure of 6 psf is used for Groups III and VI. The force is applied at the windward quarter point of the transverse superstructure.

4.1.4 Wind on Live Load

A moderate wind force is assumed to act on the live load itself, represented by a live load acting 6 feet above the roadway surface, both transversely and longitudinally. This force is computed by multiplying the bridge length tributary to a particular member by 0.1 for transverse and 0.04 for longitudinal direction.

4.1.5 Earthquake Loads

- a. Design for earthquake shall be in accordance with Division 1-A, Seismic Design of the 1996 AASHTO Standard Specifications for Seismic Design of Highway Bridges or AASHTO LRFD Specifications.
- b. The Multimode Spectral Method of dynamic analysis described in the AASHTO Specifications shall be used for most continuous bridges. The SEISAB computer program can be used to analyze most common bridges. The GTSTRUDL dynamic analysis system is capable of handling a larger range of structures.
- c. The Single Mode Spectral Method may be used in certain cases, as described in the AASHTO Specifications.
- d. Use the USGS Peak Ground Acceleration map (Appendix 4.4-A2, 10 percent Probability of Exceedance in 50 Years) to obtain an acceleration coefficient for preliminary design. The project Foundation Report will contain the acceleration coefficient to use in the final design of a bridge. When using Appendix 4.4-A2, interpolate between contours to find the value to use for particular site, and round to the nearest 1 percent of gravity (g). In general, Appendix 4.4-A2 can also be used for

bridge seismic retrofit designs. However, seismic evaluation and retrofitting of older bridges can sometimes result in excessive costs (the retrofit costs are not consistent with the benefit gained). In these situations, the Bridge Design Engineer should be consulted for direction.

- e. It is recommended that temporary (detour) structures shall be designed for a seismic acceleration coefficient equal to $0.5 \times$ the acceleration coefficient for a permanent structure. All other requirements of the AASHTO Specifications for Seismic Design of Highway Bridges shall apply. Seismic Performance Category shall be based on the magnitude of the reduced acceleration coefficient.
- f. The Geotechnical Engineer should be consulted when determining the soil type to be used in the seismic analysis.

4.1.6 Other Loads

A. Thermal, Shrinkage, and Prestressing

Member loadings are induced by movements of the structure and can result from several sources. Movements due to temperature changes are calculated using coefficients of thermal expansion of 0.000006 ft/ft per degree for concrete and 0.0000065 ft/ft per degree for steel. Reinforced concrete shrinks at the rate of 0.0002 ft/ft.

Refer to AASHTO and *Bridge Design Manual* Chapters 6, 8, and 9 for guidance on computation and application of these force types.

B. Buoyancy

The effects of submergence of a portion of the substructure is to be calculated, both for designing piling for uplift and for realizing economy in footing design.

C. Centrifugal

Centrifugal forces are included in all groups which contain vehicular live load. They act 6 feet above the roadway surface and are significant where curve radii are small or columns are long. They are radial forces induced by moving trucks. See AASHTO for force equation.

D. Collision Force on Bridge Substructure

E. Collision Force on Traffic Barrier

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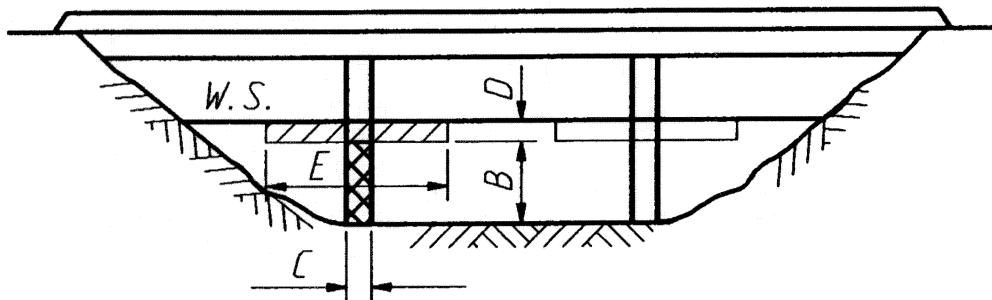
Criteria

Loads and Loading

Loads

F. Force from Stream Current, Floating Ice, and Drift

In designing for stream flow force on piers, a reasonable area of drift or floating ice must be determined, considering the stream or river characteristics (check with the Hydraulics Unit). Water depth and pier spacing will partly determine drift areas.



W.S. = Water surface as defined by the Hydraulics Unit

SF = $P_d A_d + P_p A_p$

A_d = Area of drift or floating ice = $D \times E$

A_p = Area of pier below ice = $B \times C$. Where the pier is skewed to the stream, flow C equals the width of the column normal to the stream flow.

V = Velocity of water (ft/sec)

P_d = Pressure on drift (psf) = $1.38 V^2$

P_p = Pressure on pier (psf) = $K V^2$

In the absence of other data, the maximum values of D and E shall be 10 feet and 50 feet, respectively.

Water Related Forces

Figure 4.1.6-1

DP:BDM4

BRIDGE DESIGN MANUAL

Criteria

Loads and Loading

Loads

4.2 Load Combinations

4.2.1 Combination of Loads

Group numbers represent various combinations of loads and forces which may act on a structure. Group loading combinations for both Load Factor and Service Load Design are defined by the following equation:

$$\text{Group (N)} = \gamma[\beta_d D + \beta_p PS + \beta_L (L+I) + \beta_c CF + \beta_E E + \beta_B B + \beta_s SF + \beta_w W + \beta_{WL} WL + \beta_L LF + \beta_R (R + S + T) + \beta_{EQ} EQ + \beta_{ICE} ICE]$$

where:

N	=	Group Number
γ	=	General Factor
β_N	=	Specific Factor
D	=	Dead Load (including overburden)
PS	=	Prestress Load*
L	=	Live Load
I	=	Live Load Impact
E	=	Earth Pressure (Lateral, only)
B	=	Buoyancy
W	=	Wind Load on Structure
WL	=	Wind Load on Live Load — 100 pounds per linear foot of span
LF	=	Longitudinal Force from Live Load
CF	=	Centrifugal Force
R	=	Rib Shortening
S	=	Shrinkage
T	=	Temperature
EQ	=	Earthquake
SF	=	Stream Flow Pressure
ICE	=	Ice Pressure

*PS = Forces and moments transferred from members containing post-tensioning steel to other members upon application of the post-tensioning force.

Terms in the general equation that do not contribute to a particular combination are represented by zeros in the table.

4.2.2 Load Factor Coefficients, AASHTO Standard Specifications

LFD requires basic design loads or related internal moments and forces to be increased by specified load factors, γ and β .

The γ factor is applied for stress control. Its common value is 1.3, which enables use of 77 percent of the ultimate capacity. The 30 percent increase in design load represented by the factor is intended to account for variations in weight, reinforcement placement, structural behavior, and calculation of stress.

The β factor is a measure of the accuracy of load prediction and the probability of simultaneous application of loads in a combination.

Table 4.2.2-1 contains the terms and factors required to meet AASHTO Load Factor Design.

BRIDGE DESIGN MANUAL

Criteria

Loads and Loading

Load Combinations

GROUP	β FACTORS													
	γ	D	P.S.	L+1	CF	E	B	SF	W	WL	LF	R+S+T	EQ	ICE
I	1.3	β_D	0.77	1.67	1	β_E	β_D	1	0	0	0	0	0	0
IA	1.3			2.2	0	0	0	0	0	0	0	0	0	0
II	1.3			0	0	β_E	β_D	1	1	0	0	0	0	0
III	1.3			1	1			1	0.3	1	1	0	0	0
IV	1.3			1	1			1	0	0	0	1	0	0
V	1.25		0.80	0	0			1	1	0	0	1	0	0
VI	1.25			1	1			1	0.3	1	1	1	0	0
VII	*1.0	1	1.00	0	0		1	1	0	0	0	0	1	0
VIII	1.3	β_D		1	1		β_D	1	0	0	0	0	0	1
IX	1.2		0.83	0	0			1	1	0	0	0	0	1
X	1.3	1	0.77	1.67	0		0	0	0	0	0	0	0	0

Column Design

$\beta_D = 0.75$ or $bD = 1.0$, whichever governs.

Flexural and Tension Members

$\beta_D = 1.0$

$\beta_E = 1.0$

Footing Bearing Pressure and Internal Footing Stresses

$\beta_D = 0.75$ or $\beta_D = 1.0$

$\beta_E = 1.0$

Footing Stability and Sliding

$\beta_D = 0.75$ or $\beta_D = 1.0$, whichever governs.

$\beta_E = 0.4$ or $\beta_E = 1.3$, whichever governs.

Notes:

1. For footing design, check Basic Loading Combination in accordance with BDM Section 9.5.1A3.a.
2. For rigid frame design, see BDM Section 9.3.4.E.
3. Check stability for all group loadings in accordance with BDM Section 9.5.1A3.b.
4. Group 1A load combination shall be applied only with live loadings less than HS 20 or H 20. See AASHTO.

*Applies if design loads are already factored, such as in cases where $M_{Des} = 1.0 M_L + 0.3 M_T$ or $M_{Des} = 0.3 M_L + 1.0 M_T$ are used.

Table of Coefficients γ and β

For Load Factor Design

Table 4.2.2-1

4.2.3 Service Load Coefficients, AASHTO Standard Specifications

Table 4.2.3-1 contains the terms and factors required to meet AASHTO Service Load Design. The allowable percentage of the basic unit stress is given in the right hand column of the table.

GROUP	<i>β FACTORS</i>														
	<i>γ</i>	<i>D</i>	P.S.	<i>L+1</i>	<i>CF</i>	<i>E</i>	<i>B</i>	<i>SF</i>	<i>W</i>	<i>WL</i>	<i>LF</i>	<i>R+S+T</i>	<i>EQ</i>	<i>ICE</i>	<i>%</i>
I	1.0	1	1	1	1	β_E	1	1	0	0	0	0	0	0	100
II	1.0			0	0	1	1	1	1	0	0	0	0	0	125
III	1.0			1	1	β_E	1	1	0.3	1	1	0	0	0	125
IV	1.0			1	1	β_E	1	1	0	0	0	1	0	0	125
V	1.0			0	0	1	1	1	1	0	0	1	0	0	140
VI	1.0			1	1	β_E	1	1	0.3	1	1	1	0	0	140
VII	1.0			0	0	1	1	1	0	0	0	0	1	0	*
VIII	1.0			1	1	1	1	1	0	0	0	0	0	1	140
IX	1.0			0	0	1	1	1	1	0	0	0	0	1	150
X	1.0	↓	↓	1	0	β_E	0	0	0	0	0	0	0	0	100

% = PERCENTAGE OF BASIC UNIT STRESS

CULVERT

* 150 FOR STEEL
133 FOR CONCRETE

Footing Bearing Pressure and Internal Footing Stresses

$$\beta_E = 1.0$$

Footing Stability and Sliding

$$\beta_E = 0.5 \text{ or } \beta_E = 1.0, \text{ whichever governs.}$$

Notes:

1. For culvert loading, see AASHTO.
2. No increase in allowable unit stresses shall be permitted for members or connections carrying wind load only.

Table of Coefficients γ and β
For Service Load Design
Table 4.2.3-1

BRIDGE DESIGN MANUAL

Criteria

Loads and Loading

Load Combinations

4.2.4 AASHTO LRFD

The load combinations and factors to be used for foundation design are provided in Table 4.2.4-1 and Table 4.2.4-2.

Load Combinations and Load Factors (from AASHTO LRFD Specifications Table 3.4.1-1)
Table 4.2.4-1

Load Combination Limit State	DC DD DW EH EV ES	LL IM CE BR PL LS	WA	WS	WL	FR	TU CR SH EL	TG	SE	Use One of These at a Time			
										EQ	IC	CT	CV
Strength-I	γ_p	1.75	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Strength-II	γ_p	1.35	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Strength-III	γ_p	—	1.00	1.40	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Strength-IV EH, EV, ES, DW DC only	γ_p 1.5	—	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—
Strength-V	γ_p	1.35	1.00	0.40	0.40	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Extreme Event-I	γ_p	γ_{EQ}	1.00	—	—	1.00	—	—	—	1.00	—	—	—
Extreme Event-II	γ_p	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00
Service-I	1.00	1.00	1.00	0.30	0.30	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Service-II	1.00	1.30	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—
Service-III	1.00	0.80	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
Fatigue-LL, IM and CE only	—	0.75	—	—	—	—	—	—	—	—	—	—	—

BRIDGE DESIGN MANUAL

Criteria

Loads and Loading

Load Combinations

Load Factors for Permanent Loads, γ_p

(Adapted from Table 3.4.1-2 of the AASHTO LRFD Specifications, but modified as shown below)

Table 4.2.4-2

Type of Load	Load Factor	
	Maximum	Minimum
DC: Components and Attachments	1.25	0.9
DD: Downdrag	1.00*	1.00*
DW: Wearing Surfaces and Utilities	1.50	0.65
EH: Horizontal Earth Pressure		
• Active	1.50	0.90
• At-Rest	1.35	0.90
EV: Vertical Earth Pressure		
• Retaining Structure	1.35	1.00
• Rigid Buried Structure	1.30	0.90
• Rigid Frames	1.35	0.90
• Flexible Buried Structures other than Metal Box Culverts	1.95	0.90
• Flexible Metal Box Culverts	1.50	0.90
ES: Earth Surcharge	.50	0.75

*DD was reduced to 1.00 to reflect current WSDOT and national practice.

Permanent Load Factors

DC = dead load of structural components and non structural attachments

DD = downdrag

DW = dead load of wearing services and utilities

EH = horizontal earth pressure load

EV = vertical pressure from dead load of earth fill

ES = earth surcharge load

EL = accumulated locked-in force effects resulting from the construction process

Various Transient Load Factors

BR = vehicular braking force

LS = live load surcharge

CE = vehicular centrifugal force

PL = pedestrian live load

CR = creep

SE = settlement

CT = vehicular collision force

SH = shrinkage

CV = vessel collision force

TG = temperature gradient

EQ = earthquake

TU = uniform temperature

FR = friction

WA = water load and stream pressure

IC = ice load

WL = wind on live load

IM = vehicular dynamic load allowance

WS = wind load on structure

LL = vehicular live load

The load factors γ_{TG} and γ_{SE} are to be determined on a project specific basis in accordance with Articles 3.4.1 and 3.12 of the AASHTO LRFD Specifications.

BRIDGE DESIGN MANUAL

Criteria

Loads and Loading

Load Combinations

4.3 Application of Loads

4.3.1 Dead Loads

Dead load is commonly applied to supports by assuming that it acts along each girder line.

4.3.2 Live Loads

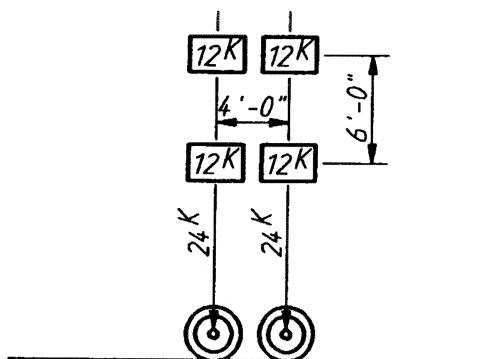
The three types of live loadings ordinarily applied to a bridge when checking for maximum stresses in its components are illustrated in AASHTO Standard Specifications and Figure 4.3.2-1. The standard HS truck represents common vehicles. The lane load consists of combinations of uniform and concentrated loads which represent three lighter trucks spaced close together. The alternative loading represents certain heavy military vehicles.

The loading type governing the design depends on the structure configuration. For example, truck loading governs for maximum moment in simple spans shorter than 145 feet and lane loading controls for longer spans. In continuous spans, lane loading governs for maximum negative moment, except for spans shorter than 45 feet, in which truck loading will govern. The maximum positive moment in continuous spans is usually produced by using lane loading, for span lengths of over about 110 feet. Alternative loading governs in certain short span situations.

Figures 4.3.2-2 and 4.3.2-3 illustrate application of loads to produce maximum stresses in various span arrangements. Appendix 4.3-B1 illustrates calculation of reactions and maximum moments in a simple span. Impact is figured using the following formula:

$$I = \frac{50}{L + 125}$$

Where L is the loaded portions of the spans.

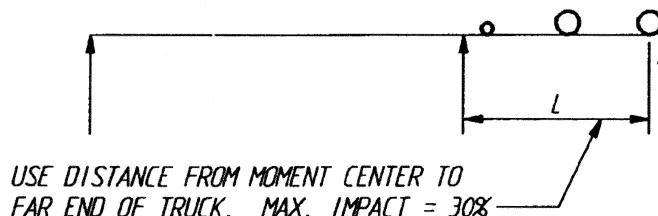


Alternative (Military) Loading
Figure 4.3.2-1

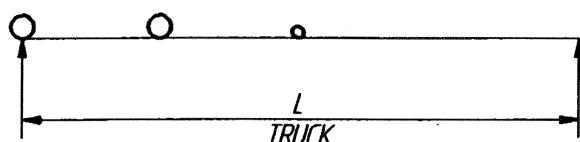
The dynamic load allowance (Impact) factor for bridges designed in accordance with the LRFD Specification shall be taken:

- IM ≈ 33% Bridge members for all limit states
- IM ≈ 15% For Fatigue & Facture limit states
- IM ≈ 75% Deck Joints for all limit states

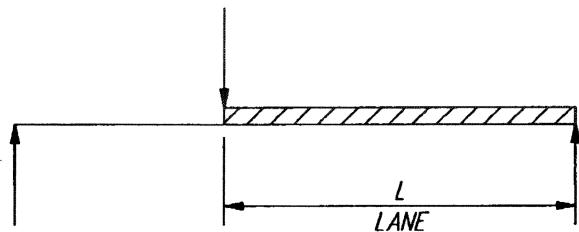
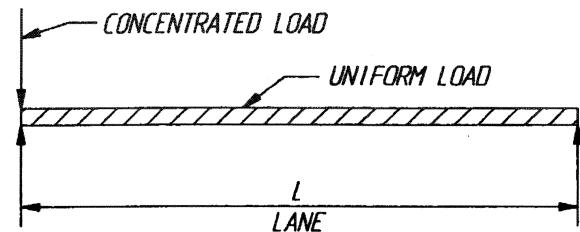
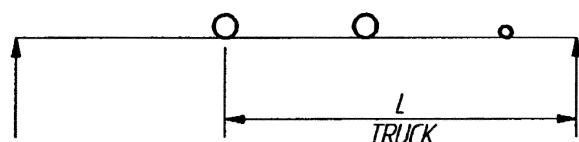
MOMENT IN CANTILEVER ARMS



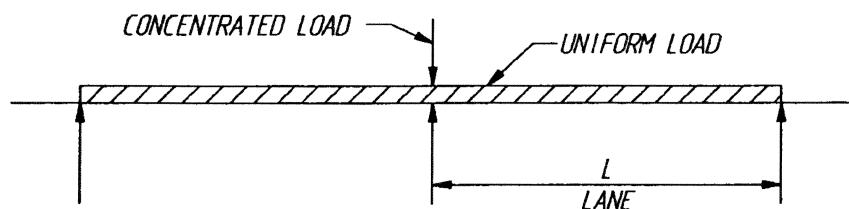
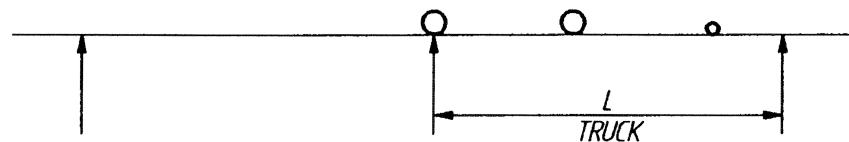
SHEAR AT SUPPORT IN SIMPLE SPANS



SHEAR WITHIN SPAN, SIMPLE SPANS



SHEAR AT SUPPORT IN CONTINUOUS SPANS



Application of Loads
Figure 4.3.2-2

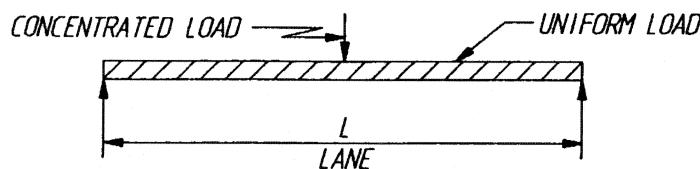
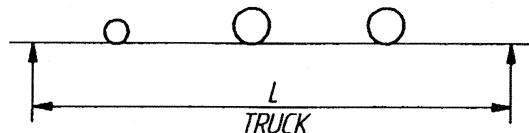
BRIDGE DESIGN MANUAL

Criteria

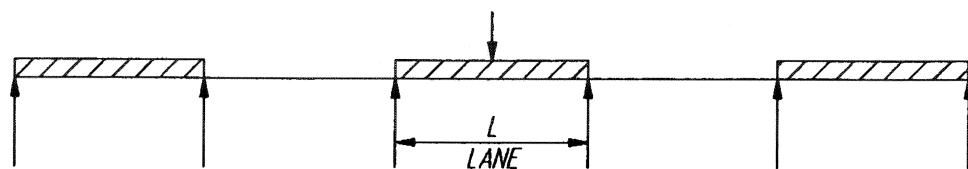
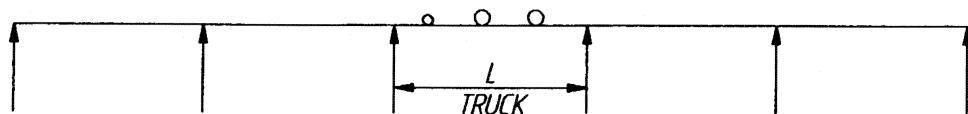
Loads and Loading

Application of Loads

ALL MOMENTS IN SIMPLE SPANS

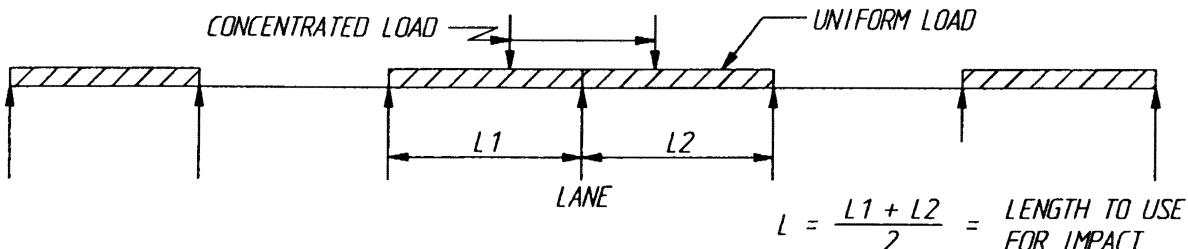
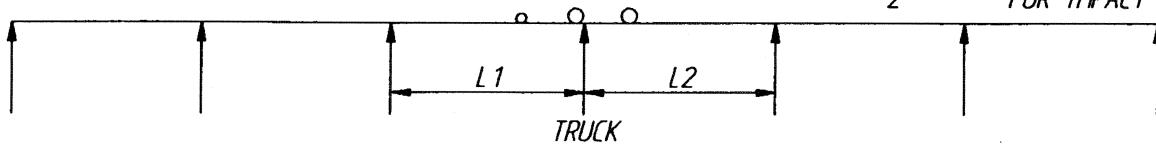


POSITIVE MOMENTS IN CONTINUOUS SPANS



NEGATIVE MOMENTS IN CONTINUOUS SPANS

$$L = \frac{L_1 + L_2}{2} = \text{LENGTH TO USE FOR IMPACT}$$



Application of Loading
Figure 4.3.2-3

4.3.3 Wind Loads

Wind loads acting on the superstructure are based on the profile presented to the wind, the height of which usually consists of the girder depth and traffic barrier height.

4.3.4 Earthquake Loads

Bibliography 1 through 4 contain several examples of applying earthquake loads to bridges. This section serves to amplify some analysis concepts.

Load factors applied in the Group VII combination are based on two concepts:

1. Full utilization of the elastic capacity of a particular element or member.
2. Taking advantage of the ductility or redundancy of the structure to absorb the energy released in an earthquake and keep the structure intact.

Two typical AASHTO load case equations are:

$$M_{EQ} = 1.0 M_L + 0.3 M_T$$

or

$$M_{EQ} = 1.0 M_L + 1.0 M_T$$

Where the moments are:

M_{EQ} = Earthquake

M_L = Longitudinal

M_T = Transverse

These equations are intended to satisfy concept 1. The SEISAB computer program prints out solutions to the two equations as load cases 3 and 4.

Concept 2 is handled through use of the "R" factor. It appears in the factored loading equation:

$$M_u = 1.0 (M_{DL} + M_{EQ}/R)$$

The Guide Specification lists values for "R" for various structural components and types of supports. Some common examples are:

- Single column bents, considered ductile but nonredundant, $R = 3$ for both directions.
- Multi-column bents, considered ductile and redundant, $R = 5$ both ways.
- Wall-type piers, less ductile than single column bents, often having $R = 2$ for transverse behavior and $R = 3$ longitudinally.
- Footings, $R = 1$ for seismic performance Categories C and D and $R = R_{col}$ for SPC B. Higher values are used than for columns and crossbeams because below ground structural damage is difficult to spot and repair. Plastic hinging moments are often less than those produced using an R of 1, so that some economy may be realized.
- Bearing type connections and stops, $R = 0.8$, due to lack of ductility and redundancy and because they serve to prevent large displacements.

See Appendix 4.3-B2-1 and 2 for illustrations of common piers and appropriate factors to apply to the members.

In order to design structures to survive the forces and strains resulting from earthquake motion, the following factors need to be considered:

- The proximity of the site to known active faults and the historical record of activity.
- The seismic response of the soil at the site.
- The dynamic response characteristics of the total structure.

See Appendix 4.3-B3-1 through 3 for a general discussion of a seismic analysis.

4-3:P:BDM4

BRIDGE DESIGN MANUAL

Criteria

Loads and Loading

Application of Loads

4.4 Foundation Modeling

Proper foundation modeling for earthquake loads is necessary because misinterpreted AASHTO Specifications can lead to a wide range of member sizes. Realistic models will likely produce savings in material, especially when determining loads to apply to a substructure. Analysis is an iterative process which converges to an acceptable design.

4.4.1 Procedure Summary

Following is a workable procedure for analysis:

- a. Assume the foundation as fixed (unless you know otherwise). Use SEISAB or GTSTRU DL to perform a dynamic analysis to determine initial loading.
- b. If the support is not founded in rock, multiply the forces from the fully fixed model by 0.85 for the initial trial design. Otherwise, use the fully fixed forces for the trial.
- c. Determine a preliminary footing size, pile size, and arrangement, as applicable to the type of support.
- d. Determine foundation springs as outlined in this section and Section 4.4.2. If pile support is being used, see Section 4.4.3.E.
- e. Rerun the dynamic model with springs included.
- f. Compare loads and deflections using the same range used to determine the springs.
- g. Redesign the footing, piles, adjust the springs, etc., until tolerable convergence is attained.

4.4.2 Spread Footings

- a. You may apply load factor column moments from groups other than Group VII and column plastic hinging moments for a first trial footing configuration. Then determine soil spring constants using the footing plan area and depth of embedment. Assuming a shear wave velocity value, consult a Foundation or Geotechnical Engineer for an appropriate value.
- b. Appendix 4.4-B1 through 4 illustrate a procedure to determine soil spring constants for spread footings.

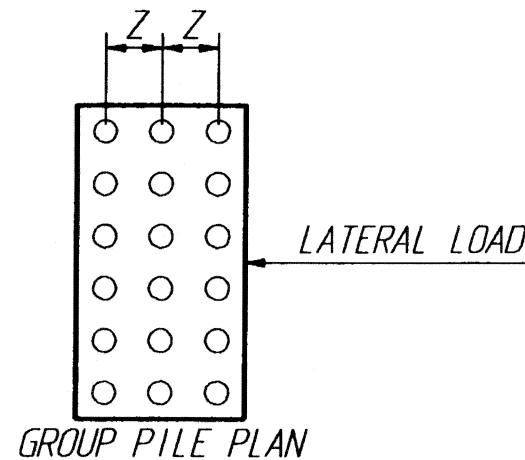
4.4.3 Pile Foundations**A. Lateral Spring Input from P-Y Curves**

Spring constants that represent pile supports may be obtained using a procedure which begins by applying moments (as described in Section 4.4.1A) to an assumed footing and pile configuration. P-Y curves from the foundation report may be input to the LPILE1 computer program to derive the initial spring constants.

The spacing between pile centers is often about 4 times the pile diameter (D), which means that each pile in the group may deflect more than if it were acting alone. Apply efficiency factors, if provided on the soils report, to quantify that difference. If information is not available, use the following table to estimate values.

Z	EFFICIENCY (REDUCTION) FACTOR
8D	1.0
6D	0.8
4D	0.5
3D	0.4

EFFICIENCY FACTOR



Efficiency Factor
Table 4.4.3-1

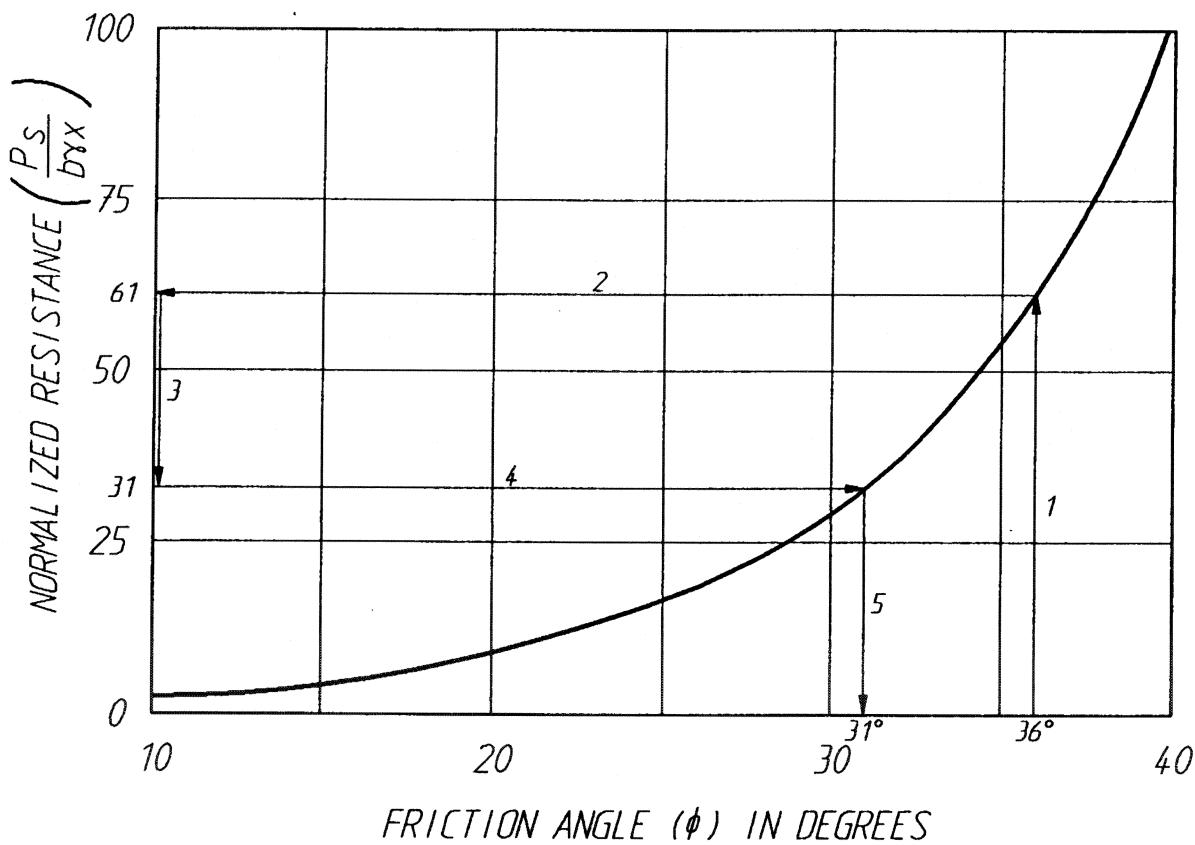
For driven piles, the following factors apply:

Contact the Olympia Service Center Materials Lab to verify any assumptions.

The LPILE1 computer program will generate P-Y curves, or the user can input them. To obtain generated curves, input a modulus of subgrade reaction (K), and a soil shear strength (C) which are the values taken from the soils report multiplied by the efficiency factor. To figure P-Y curves for input, multiply the P-Y values from the soils report by the efficiency factor.

For a typical soil, the relationship between its normalized resistance value and friction angle is defined by the curve in Figure 4.4.3-1. The friction angle could be adjusted for efficiency and input to LPILE1 by following these steps:

1. Begin at the coordinate of the natural friction angle (36°).
2. Read across to the normalized resistance (61).
3. Multiply the resistance by the efficiency reduction factor, i.e., $61 (0.5) = 31$.
4. Read across from the reduced value to obtain the adjusted friction angle (31°).
5. Input the ϕ value to LPILE1.

Friction Angle (ϕ)

$$\frac{P_s}{bgx} = K_a (\tan^8 B - 1) + K_o \tan \phi \tan^4 B$$

P_s = Soil Resistance on Pile Element

b = Pile Width

g = Soil Unit Weight

X = Depth to Pile Element

N = Step in Example

B = $45^\circ + \phi/2$

K_a = $\tan 2(45^\circ - \phi/2)$

K_o = $1 - \sin \phi$

Figure 4.4.3-1

B. Lateral Spring Input to Dynamic Analysis

Lateral spring constants can be generated for input to SEISAB (or GTSTRU_DL) by using LPILE1 and two types of loading.

Case 1 — Applied Lateral Load — See Figure 4.4.3-2(A). Apply a lateral load (F) to the model of a pile, and restrain its top against rotation. The load produces a deflected shape with the top deflection being Δ . A moment (M) is also induced. F and M may be plotted against Δ to produce two curves. The spring constants are defined as slopes of the curves, and their calculation and SEISAB nomenclature are given by the equations in Figure 4.4.3-2(A).

Make enough LPILE1 runs to define a linear range along the lateral force versus a deflection curve. Vary axial loads, to bracket the values expected from the dynamic analysis (i.e., SEISAB results). Include negative axial loads to represent anticipated tension due to uplift effects.

Case 2 — Applied Moment — See Figure 4.4.3-2(B). Apply a moment (M) to the pile model, restraining the pile top against translation. Calculate the pile top rotation (ϕ) from the LPILE1 output by dividing the deflection at the bottom of the top increment (Δ_1) by the increment length (H_1). The spring constants are defined as slopes of the curves, and they are calculated using the equations in Figure 4.4.3-2(B).

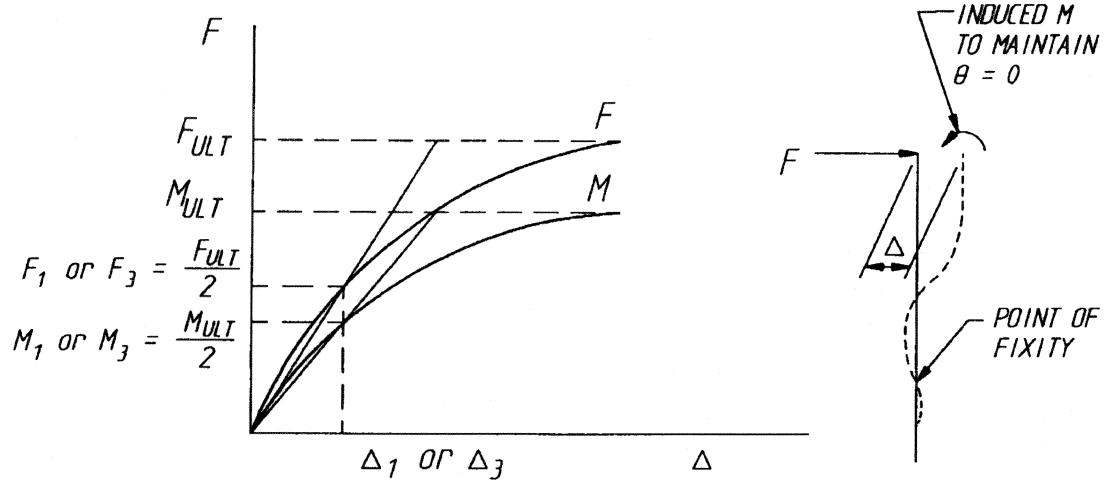
A rapid way to approximate the slope of any curve is to select a point at half of the ultimate lateral force or moment capacity of the pile. Note that the off-diagonal terms must be equal and opposite in sign.

Figure 4.4.3-3 contains examples of spring calculation from LPILE1 output.

Criteria

Loads and Loading

Foundation Modeling



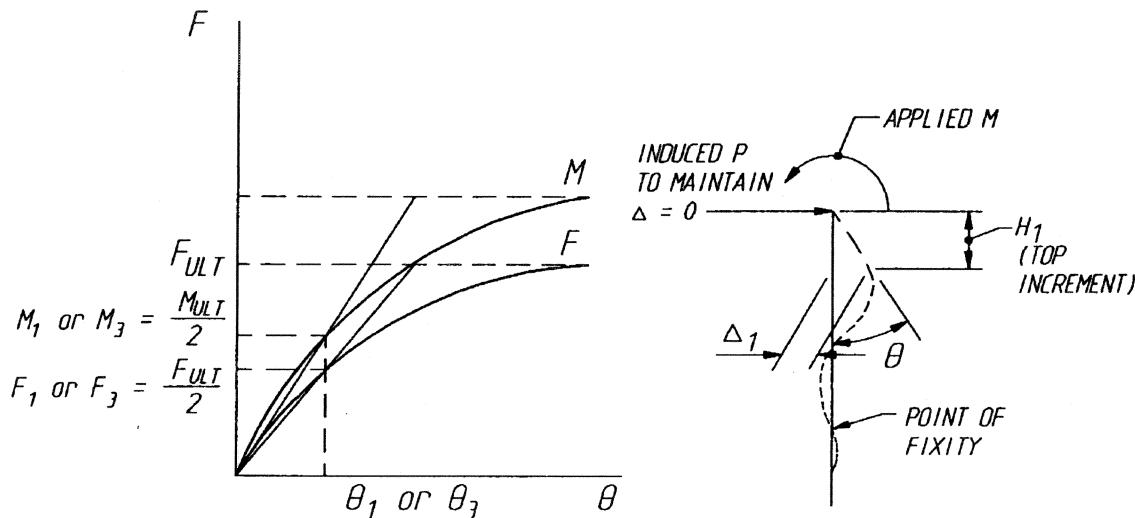
$$K F_1 F_1 = F_1 / \Delta_1 \quad K F_3 F_3 = F_3 / \Delta_3$$

(DIAGONAL TERMS IN 6×6 MATRIX)

$$K M_3 F_1 = -K M_1 F_3 = M_3 / \Delta_1 = -M_1 / \Delta_3$$

(OFF DIAGONAL TERMS)

Figure 4.4.3-2A



$$K M_1 M_1 = M_1 / \theta_1 \quad K M_3 M_3 = M_3 / \theta_3$$

(DIAGONAL TERMS IN 6×6 MATRIX)

$$-K F_3 M_1 = K F_1 M_3 = -F_3 / \theta_1 = F_1 / \theta_3$$

(OFF DIAGONAL TERMS)

Figure 4.4.3-2B

BRIDGE DESIGN MANUAL

Criteria

Loads and Loading

Foundation Modeling

Loading Number 1

Boundary condition code = 2
 Lateral load at the pile head = 0.250D+05 lbs = 25 K applied
 Slope at the pile head = 0.000D+00 in/in
 Axial load at the pile head = 0.758D+05 lbs

X	Deflection	Moment	Shear	Soil Reaction	Total Stress	Flexural Rigidity
In	In	Lbs-In	Lbs	Lbs/In	Lbs/In**2	Lbs-In**2
*****	*****	*****	*****	*****	*****	*****
0.00	0.267D+01	-0.383D+07	0.250D+05	0.000D+00	0.270D+05	0.392D+11
	=2.67"		=25 ^K			

$$KF1F1 = KF3F3 = \frac{25^K}{(2.67\text{in}/12\text{in}/\text{ft})} = 112 \frac{\text{K}}{\text{ft}}$$

(A)

Loading Number 1

Boundary condition code = 4
 Deflection at the pile head = 0.000D+00 in
 Moment at the pile head = 0.391D+07 in-lbs = 391 K-in applied
 Axial load at the pile head = 0.103D+06 lbs

X	Deflection	Moment	Shear	Soil Reaction	Total Stress	Flexural Rigidity
In	In	Lbs-In	Lbs	Lbs/In	Lbs/In**2	Lbs-In**2
*****	*****	*****	*****	*****	*****	*****
0.00	0.000D+00	0.391D+07	0.189D+05	0.000D+00	0.281D+05	0.392D+11
28.04	-0.237D+00	0.340D+07	-0.186D+05	0.208D+02	0.247D+05	0.392D+11

$$0.237" = \Delta_1$$

$$28.04" = H$$

$$f = \tan^{-1} \frac{\Delta_1}{H_1} = \tan^{-1} \frac{0.237}{28.04} = 0.48426^\circ$$

$$\text{or} = 0.00845 \text{ rad}$$

(B)

Sample LPILE1 Output
 Figure 4.4.3-3

C. Vertical Springs

Vertical spring constants, K_v (or KF_2F_2) can be calculated from the following equations:

$$\text{Point bearing pile: } K_v = \frac{AE}{L}$$

where,

- A = Cross sectional area
- E = Young's modulus
- L = Length

Pile having constant skin friction:

$$K_v = \frac{2AE}{L}$$

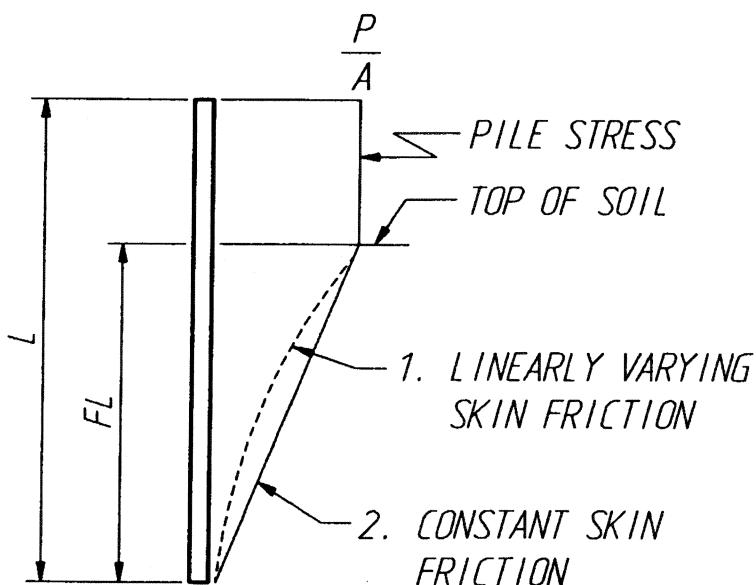
Pile linearly varying skin friction:

$$K_v = \frac{3AE}{L}$$

Pile partially embedded in the soil:

$$1. \quad K_v = \frac{AE}{\left(1 - \frac{F}{2}\right)L}$$

$$2. \quad K_v = \frac{AE}{\left(1 - \frac{2F}{3}\right)L}$$



Torsional (M/ϕ) spring constants for individual piles are based on the strength of the pile only. The torsional resistance is given by the following equation:

$$M/\phi = T/\phi = JG/L$$

where,

$$G = 0.4 E$$

$$J = \text{Torsional Moment of Inertia}$$

$$L = \text{length of pile}$$

D. Stiffness Matrix

Eight individual pile stiffness terms should be put into Seisab, which forms a $\{6 \times 6\}$ matrix as shown below:

	F1	F2	F3	M1	M2	M3
F1	KF1F1	0	0	0	0	KF1M3
F2		KF2F2	0	0	0	0
F3			KF3F3	-KF3M1	0	0
M1				KM1M1	0	0
M2					KM2M2	0
M3	"Symmetrical"					KM3M3

KF1M3 is cross-coupling term P/ϕ . -KF3M1 is cross-coupling term M/d . Note that the two have opposite signs.

E. GPILE Computer Program

If a large number of piles is required per footing, to reduce Seisab input/output, individual springs can be used in the GPILE computer program. The output will contain a $\{6 \times 6\}$ stiffness matrix for the pile group which can be used to model the foundation in SEISAB. GPILE input includes pile configuration and spring constants. The program also computes individual pile loads and deflections from a set of input loads. GPILE can be used in conjunction with the plastic hinging moments, transmitted from the column, to converge on an acceptable pile configuration.

4.99 Bibliography

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BRIDGE DESIGN MANUAL

Criteria

Loads and Loading

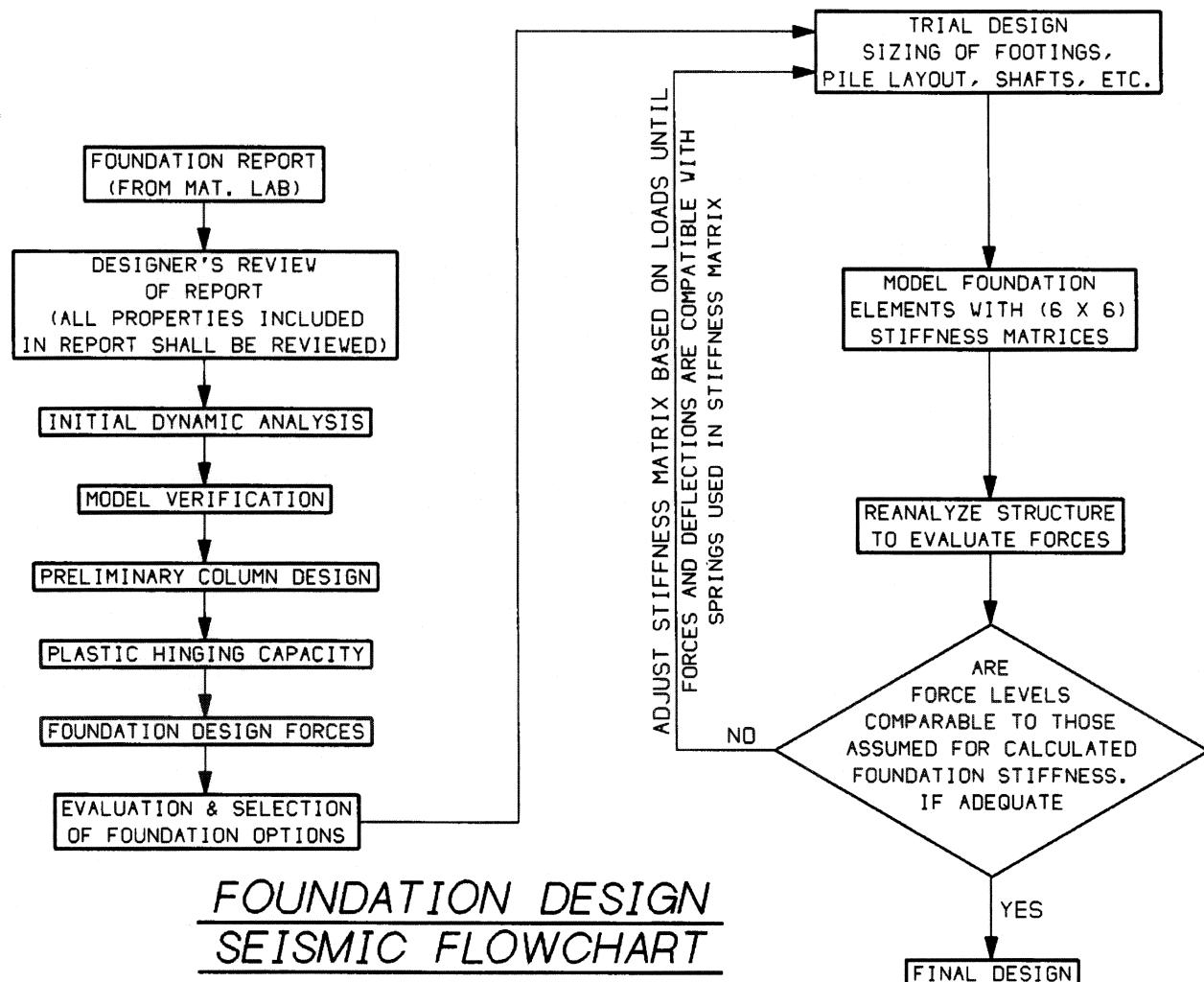
Bibliography

BRIDGE DESIGN MANUAL

Appendix A

Loads and Loading

Foundation Design Seismic Flow Chart

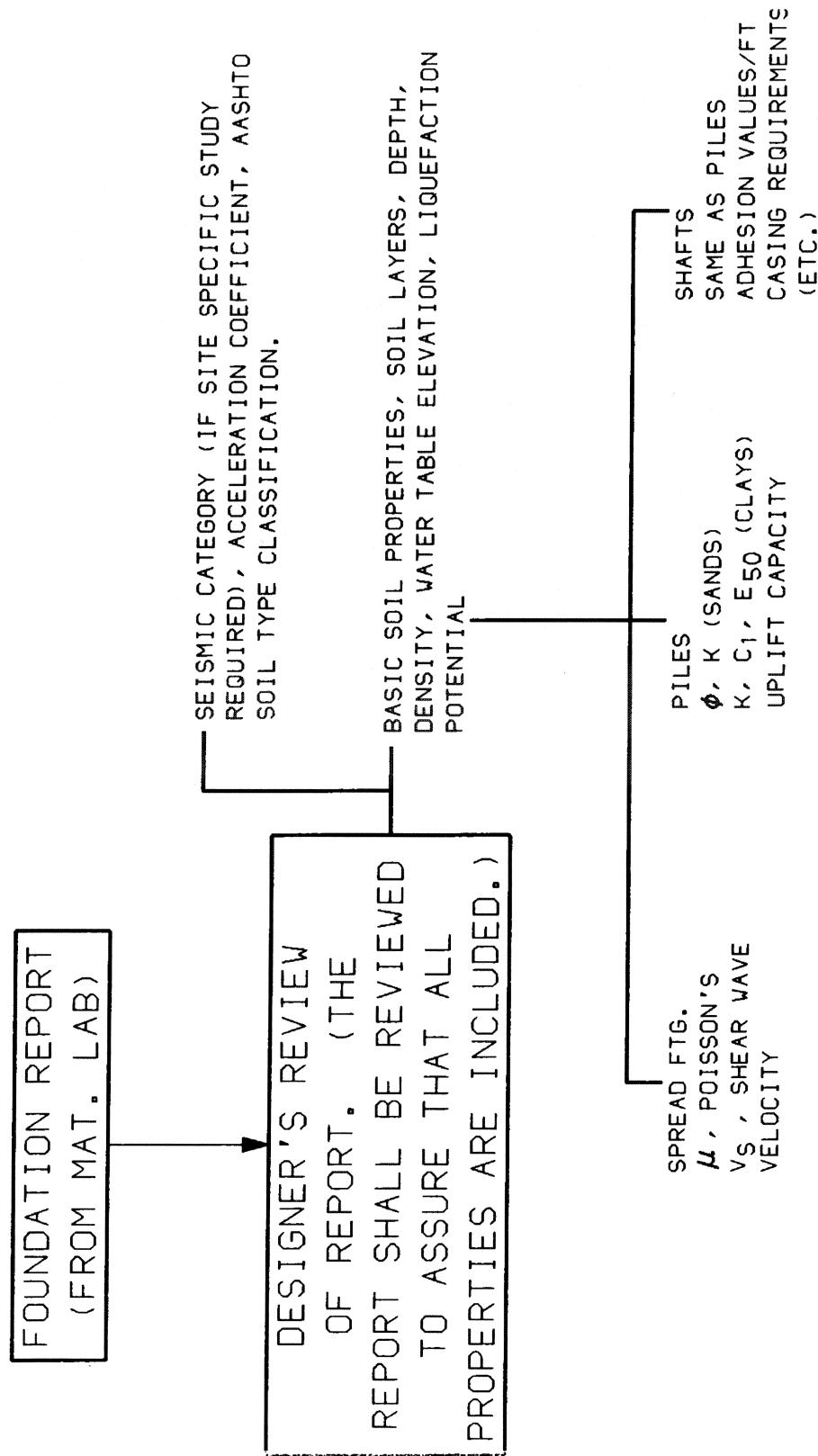


BRIDGE DESIGN MANUAL

Appendix A

Loads and Loading

Foundation Design Seismic Flow Chart

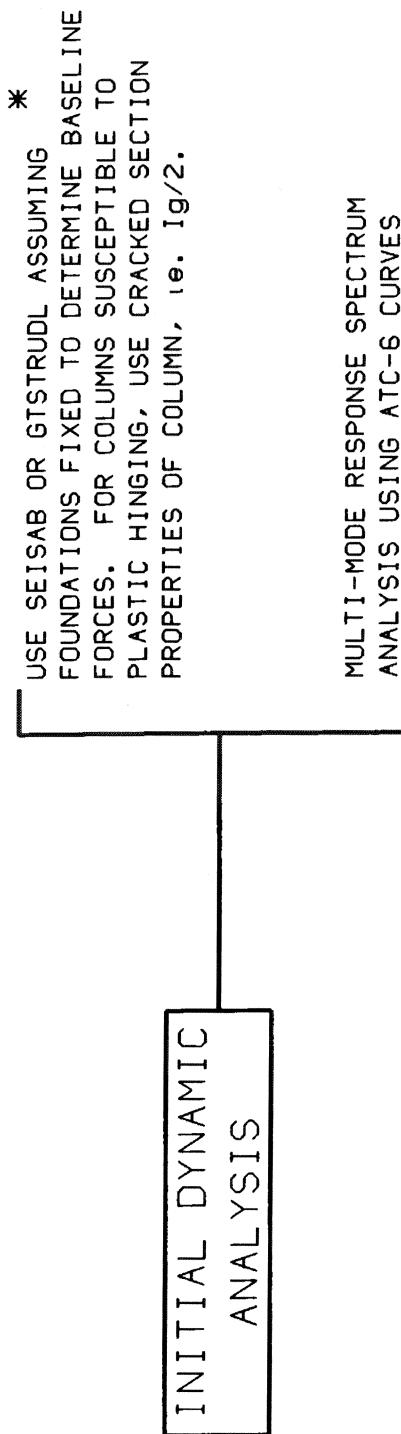


BRIDGE DESIGN MANUAL

Appendix A

Loads and Loading

Foundation Design Seismic Flow Chart



* LINEAR ELASTIC RESPONSE SPECTRUM ANALYSIS PROGRAMS AVAILABLE

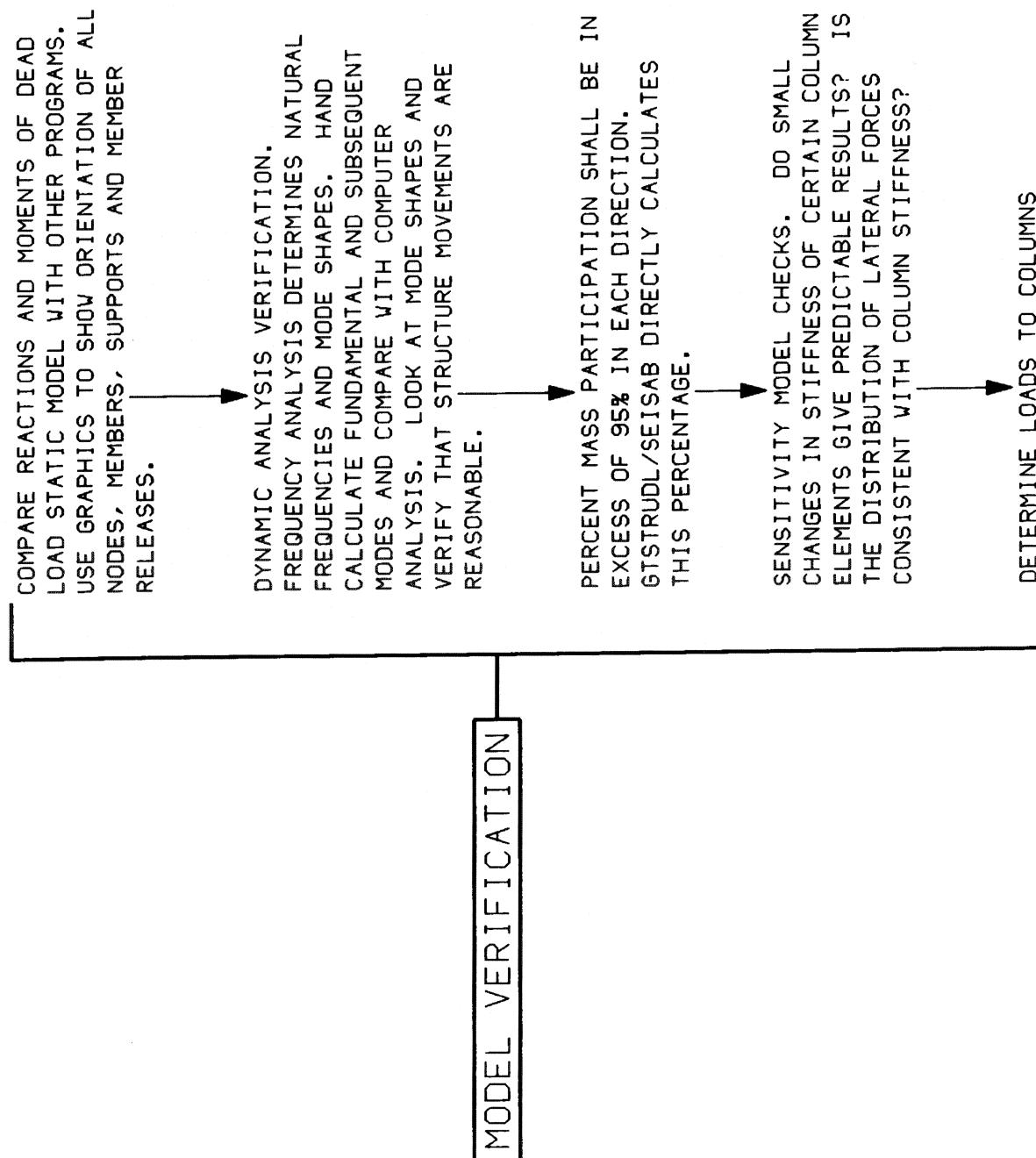
PROS	CONS
SEISAB - MODELLER - MINIMAL INPUT/OUTPUT - DYNAMIC ANALYSIS COMMANDS ARE INTEGRATED IN MODULES - SLAVED CROSSBEAM ELEMENTS	- NO GRAPHICS - SINGLE LINE BEAM ELEMENT - WON'T ALLOW DIVERGENT ALIGNMENTS - MODELLING COLUMN TOP PIN HINGE DIAPHRAGMS LOCATION PROBLEM
GTSTRU DL - MODELLER/GRAFPHICS - FINITE ELEMENT CAPABILITY FOR LARGE WIDTH/SPAN RATIO - MULTI-PURPOSE, DIVERGING ALIGNMENTS	- BETA ANGLES FOR COLUMN ELEMENTS - CROSS BEAM STIFFNESS NOT SLAVED TO SUPERSTRUCTURE - DYNAMIC ANALYSIS COMMANDS COMPLEX ~ (DIRECTIONAL FACTORS) - IMPORT RESPONSE SPECTRUM FILES

BRIDGE DESIGN MANUAL

Appendix A

Loads and Loading

Foundation Design Seismic Flow Chart

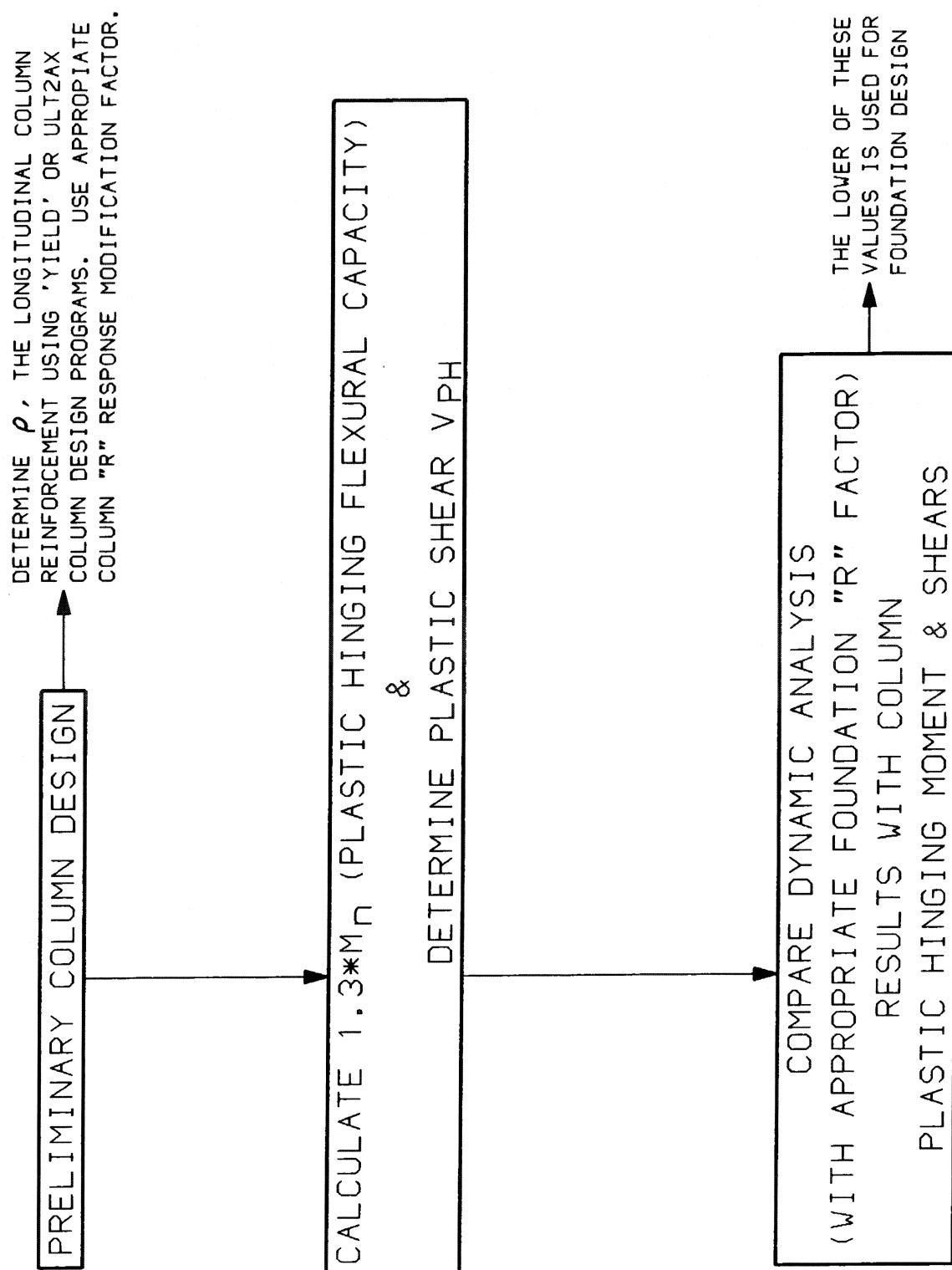


BRIDGE DESIGN MANUAL

Appendix A

Loads and Loading

Foundation Design Seismic Flow Chart

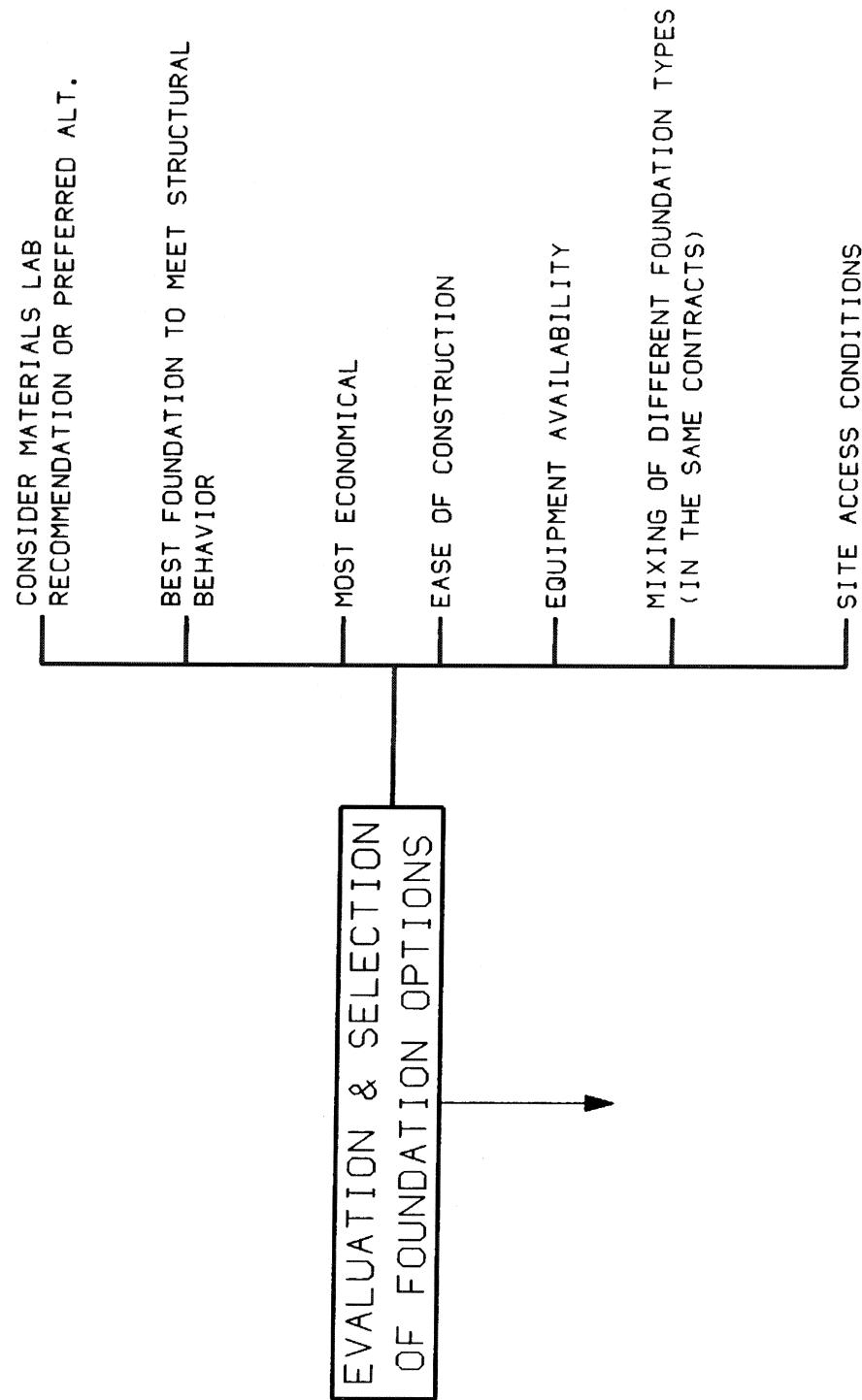


BRIDGE DESIGN MANUAL

Appendix A

Loads and Loading

Foundation Design Seismic Flow Chart

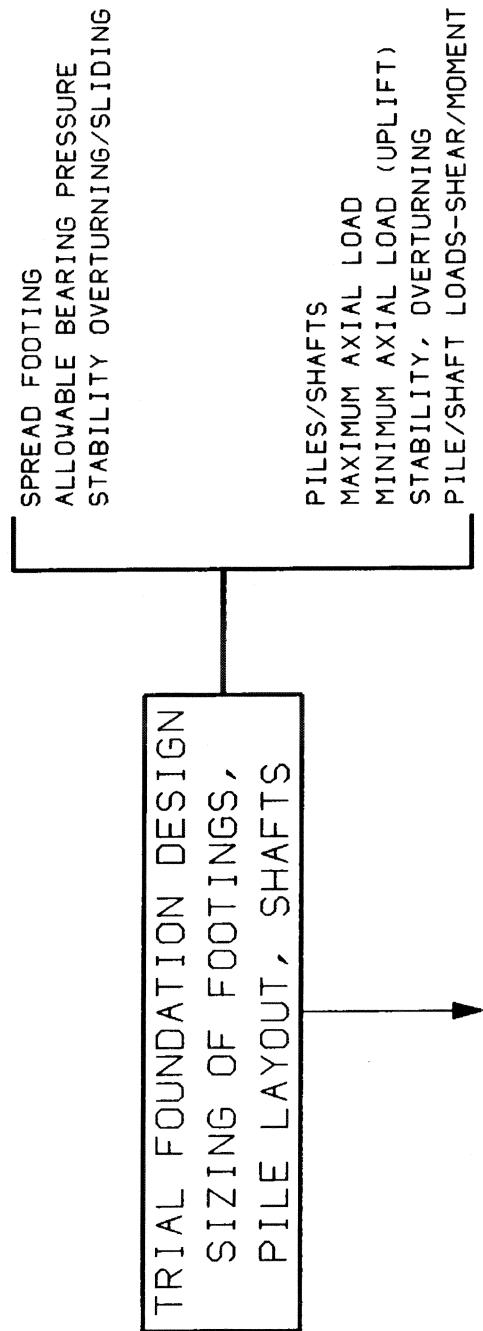


BRIDGE DESIGN MANUAL

Appendix A

Loads and Loading

Foundation Design Seismic Flow Chart

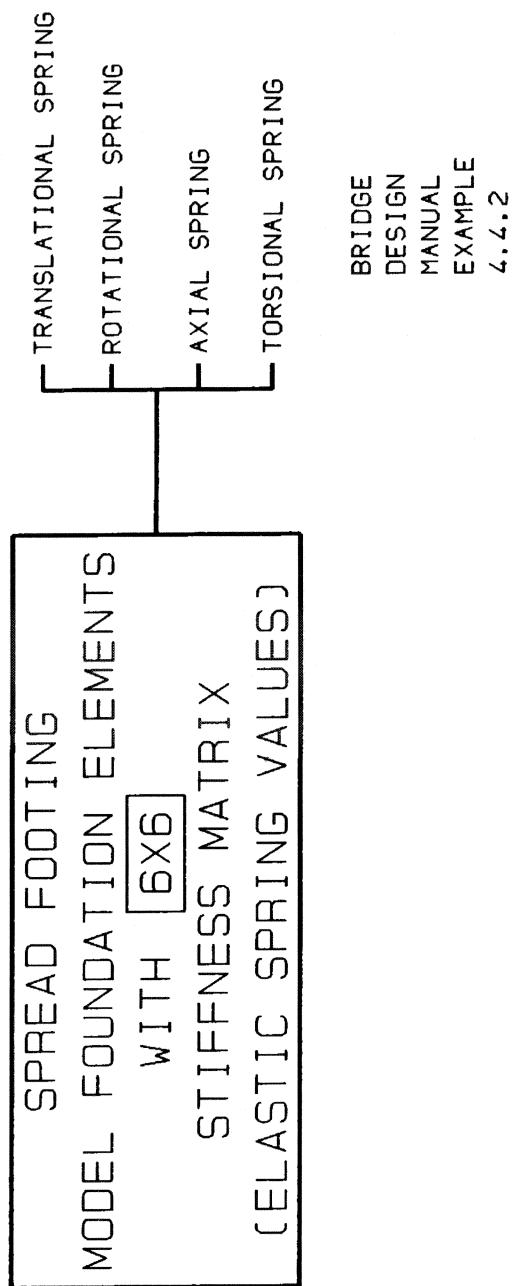


BRIDGE DESIGN MANUAL

Appendix A

Loads and Loading

Foundation Design Seismic Flow Chart



BRIDGE DESIGN MANUAL

Appendix A

Loads and Loading

Foundation Design Seismic Flow Chart

SOIL PROPERTIES ARE NON-LINEAR BY NATURE (i.e. P-Y CURVES). BY DETERMINING FORCE LEVELS ON PILES, AN APPROPRIATE LINEAR SPRING IS DEVELOPED AND CAN BE USED AS INPUT INTO THE LINEAR ELASTIC DYNAMIC ANALYSIS. AS FORCE LEVELS CHANGE, THE LINEAR SPRINGS CHANGE ACCORDINGLY, RESULTING IN AN ITERATIVE PROCESS.

DEVELOP INDIVIDUAL PILE
[6X6] STIFFNESS MATRIX-AXIAL,
LATERAL, ROTATIONAL, TORSIONAL
& CROSS COUPLING STIFFNESSES

FOR A PILE GROUP, ASSIMILATE GLOBAL
[6X6] GROUP STIFFNESS MATRIX FROM
INDIVIDUAL PILE STIFFNESS MATRICES
USING GPILE.

PILE/SHAFT FOOTINGS
MODEL FOUNDATION ELEMENTS
WITH [6X6]
STIFFNESS MATRIX
(ELASTIC SPRING VALUES)

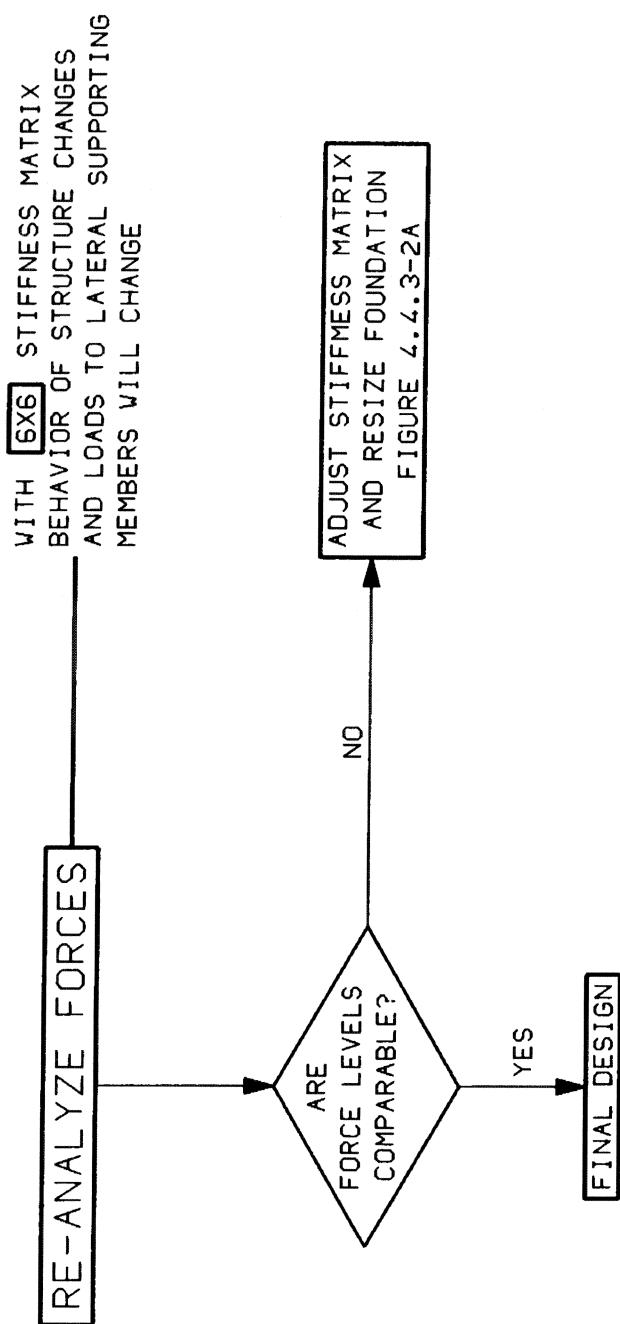
SEE BRIDGE DESIGN MANUAL
4.4.3 PILE FOUNDATIONS FOR
DETAILED DESCRIPTION OF
PROCEDURES

BRIDGE DESIGN MANUAL

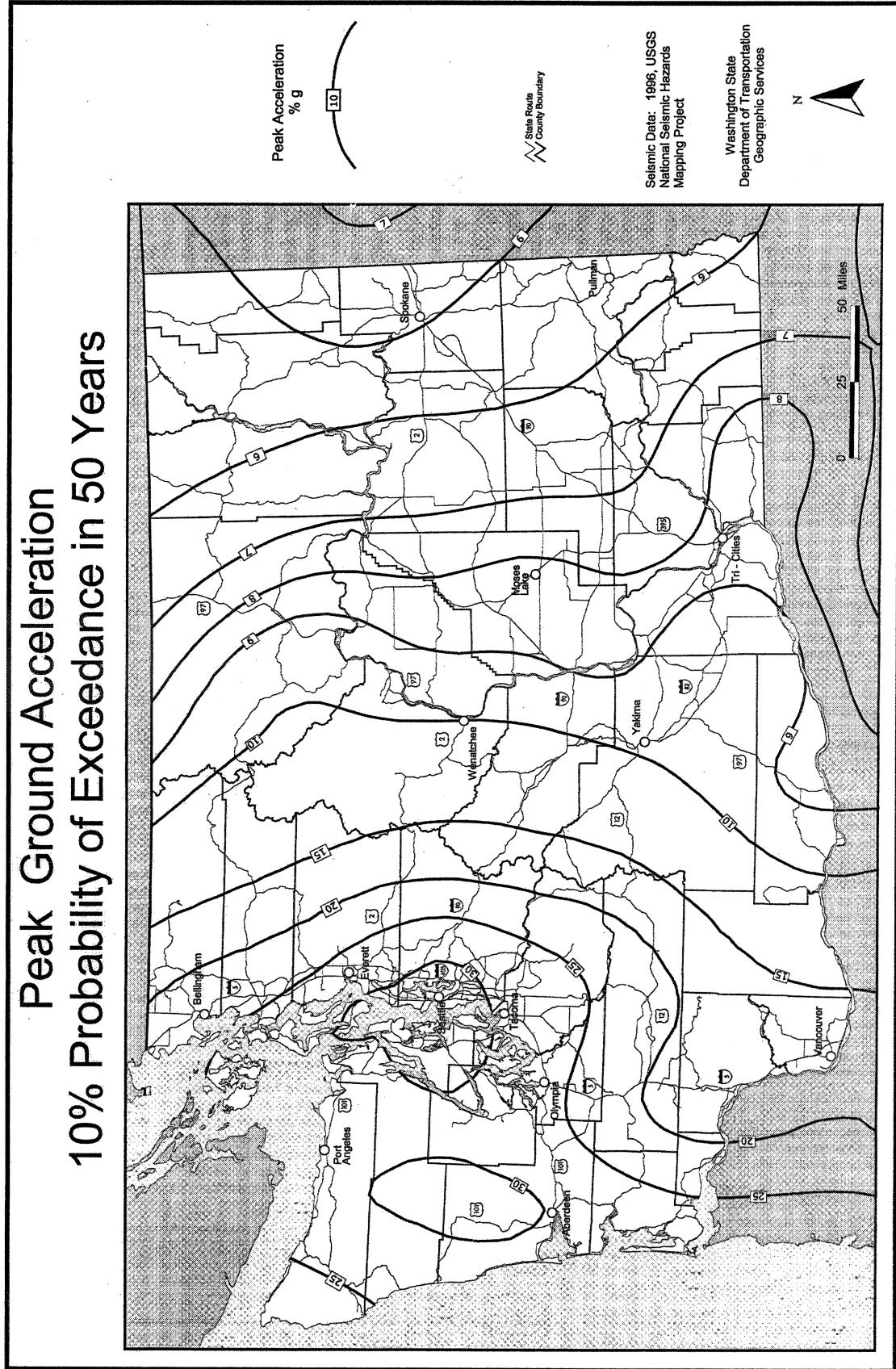
Appendix A

Loads and Loading

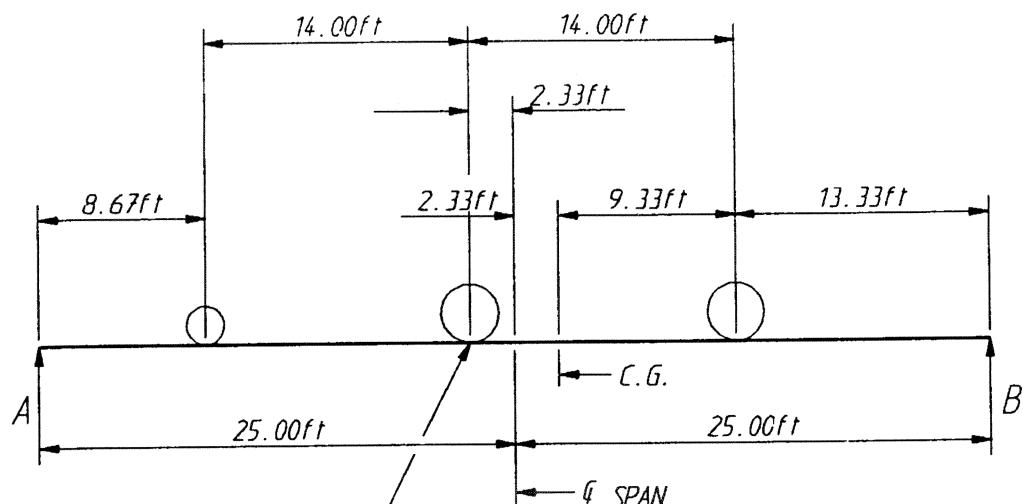
Foundation Design Seismic Flow Chart



Peak Ground Acceleration 10% Probability of Exceedance in 50 Years



Appendix B

Loads and LoadingBasic Truck Loading

THE MAXIMUM MOMENT IS UNDER THE
REAR AXLE OF THE TRUCK.

$$\text{REACTION AT } A = \frac{(13.33' + 27.33')}{50'} \times 40k + \frac{41.33'}{50'} \times 10k = 40.9k$$

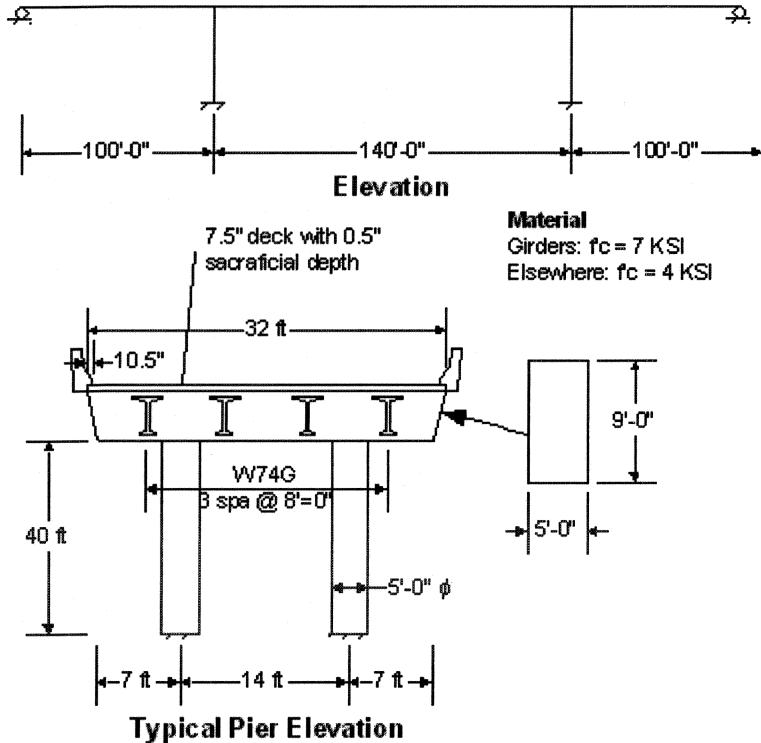
$$\text{MOMENT} = 40.9k \times 22.67' - 10k \times 14' = 787'k$$

Basic Truck Loading
HS25

1 Introduction

The purpose of this example is to demonstrate a methodology of analyzing a bridge pier for the HL-93 live load. This analysis consists of two plane frame analyzes. The first analysis is a longitudinal analysis of the superstructure. This analysis produces reactions at the intermediate piers, which are applied to a plane frame model of the pier.

2 Bridge Description



3 Analysis Goals

The purpose of this analysis is to determine the following live load actions in the top and bottom of the column and in the footing:

- Maximum axial force and corresponding moments
- Maximum moments and corresponding axial force
- Maximum shears

Additionally the following live load actions will be computed for controlling design points in the cross beam

- Maximum moment
- Maximum shear

4 Material Properties

Let's begin the analysis by determining the material properties.

Loads and Loading**HL 93 Loading for Bridge Piers**

Code Reference

4.1 Girders

$$E_c = 33,000 w_c^{1.5} \sqrt{f'_c}$$

$$w_c = 0.160 \text{ KCF}$$

$$f'_c = 7 \text{ KSI}$$

$$E_c = 33,000(0.160)^{1.5} \sqrt{7} = 5588 \text{ KSI}$$

4.2 Slab, Columns and Cross Beam

$$E_c = 33,000 w_c^{1.5} \sqrt{f'_c}$$

$$w_c = 0.160 \text{ KCF}$$

$$f'_c = 4 \text{ KSI}$$

$$E_c = 33,000(0.160)^{1.5} \sqrt{4} = 4224 \text{ KSI}$$

5 Section Properties

Compute the geometric properties of the girder, columns, and cap beam.

5.1 Girder

The composite girder section properties can be obtained from the Section Properties Calculator in QConBridge™.

$$A = 1254.6 \text{ in}^2$$

$$I = 1007880 \text{ in}^3$$

5.2 Column

Properties of an individual column can be obtained by simple formula

$$A = \pi \frac{d^2}{4} = \pi \frac{(5 \text{ ft} \cdot 12 \frac{\text{in}}{\text{ft}})^2}{4} = 2827 \text{ in}^2$$

$$I = \pi \frac{d^4}{64} = \pi \frac{(5 \text{ ft} \cdot 12 \frac{\text{in}}{\text{ft}})^4}{64} = 636172 \text{ in}^4$$

For longitudinal analysis we need to proportion the column stiffness to match the stiffness of a single girder line. Four girder lines framing into a two column bent produce a rotation and axial deflection under a unit load, the stiffness of the column member in the longitudinal analysis model needs to be 25% of that of the bent to produce the same rotation and deflection under 25% of the load.

For longitudinal analysis the section properties of the column member are

$$A = \frac{(2 \text{ columns})(2827 \text{ in}^2 \text{ per column})}{4 \text{ girder lines}} = 1413 \text{ in}^2$$

$$I = \frac{(2 \text{ columns})(636172 \text{ in}^4 \text{ per column})}{4 \text{ girder lines}} = 318086 \text{ in}^4$$

NOTE

For columns of other shapes, and for skewed bents, the properties of the columns need to be computed in the plane of the longitudinal frame, and the plane of the bent for use in each analysis respectively.

5.3 Cap Beam

Cap beam properties can also be obtained by simple formula

$$A = w \cdot h = 5 \text{ ft} \cdot 9 \text{ ft} \cdot 144 \frac{\text{in}^2}{\text{ft}^2} = 64935 \text{ in}^2$$

$$I = \frac{1}{12} w \cdot h^3 = \frac{1}{12} \cdot 5 \text{ ft} \cdot (9 \text{ ft})^3 \cdot 20736 \frac{\text{in}^4}{\text{ft}^4} = 6283008 \text{ in}^4$$

6 Longitudinal Analysis

The purpose of this analysis, initially, is to determine the maximum live load reactions that will be applied to the bent. After a transverse analysis is performed, the results from this analysis will be scaled by the number of loaded lanes causing maximum responses in the bent and distributed to individual columns.

The longitudinal analysis consists of applying various combinations of design lane and design trucks. The details can be found in LRFD 3.6

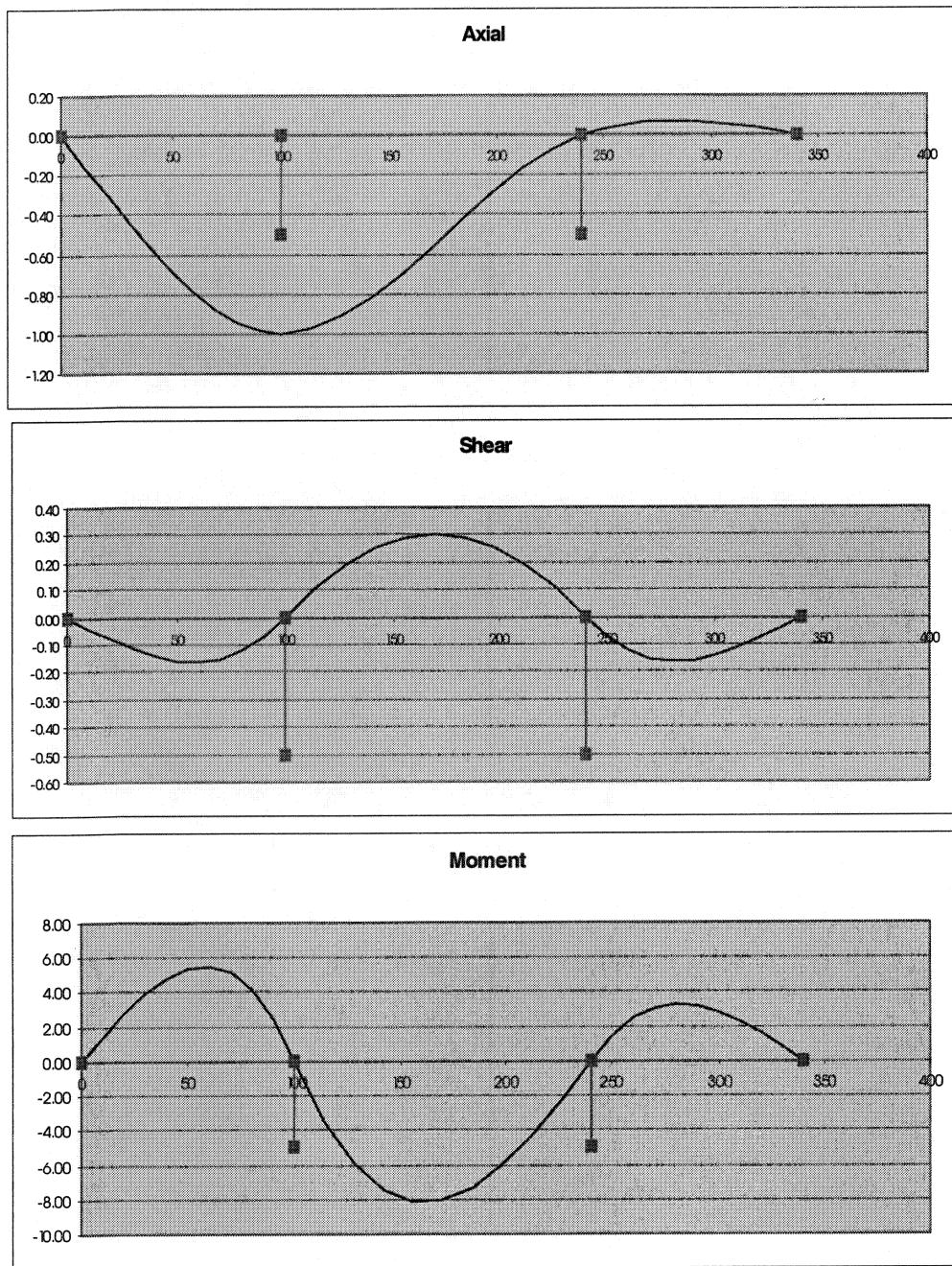
3.6

6.1 Loading

Now comes the tricky part. How do you configure and position the design vehicles to produce maximum reactions? Where do you put the dual truck train, and what headway spacing do you use to maximize the desired force effects? If we look at influence lines for axial force, moment, and shear at the top and bottom of the column, the loading configuration becomes apparent.

6.1.1 Influence Lines

The figures below are influence lines for axial force, shear, and moment at the top of Pier 2 for a unit load moving along a girder line. The influence lines for the bottom of the pier will be exactly the same, except the moment influence will be different by an amount equal to the shear times the pier height.



To achieve the maximum compressive reaction, the lane load needs to be in spans 1 and 2, and the dual truck need to straddle the pier and be as close to each other as possible. That is, the minimum headway spacing of 50 feet will maximize the compressive reaction.

Maximum shears and moments occur under two conditions. First, spans 1 and 3 are loaded with the lane load and the dual truck train. The headway spacing that causes the maximum response is in the range of 180 – 200 feet. Second, a span 2 is loaded with the lane load and the dual truck train. The headway spacing is at its minimum value of 50 ft.

Appendix B**Loads and Loading****HL 93 Loading for Bridge Piers**

Code Reference

Analytically finding the exactly location and headway spacing of the trucks for the extreme force effects is possible, but hardly worth the effort. Structural analysis tools with a moving load generator, such as GTSTRUDL™, can be used to quickly determine the maximum force effects.

6.2 Results

A longitudinal analysis is performed using GTSTRUDL™. The details of this analysis are shown in Appendix A.

The outcome of the longitudinal analysis consists of dual truck train and lane load results. These results need to be combined to produce the complete live load response.

The complete response is computed as

$$Q_{LL+IM} = 0.9[(IM)(Dual\ Truck\ Train) + Lane\ Load]$$

The dynamic load allowance (impact factor) is given by the LRFD specifications as 33%. Note that the dynamic load allowance need not be applied to foundation components entirely below ground level. This causes us to combine the dual truck train and lane responses for cross beams and columns differently than for footings, piles, and shafts.

6.2.1 Combined Live Load Response

The tables below summarize the combined live load response. The controlling load cases are given in parentheses.

Maximum Axial

		Top of Pier	Bottom of Pier
	Axial (K/LANE)	Corresponding Moment (K-FT/LANE)	Corresponding Moment (K-FT/LANE)
Dual Truck Train	-117.5 (Loading 1014)	-188.5	140.9
Lane Load	-88.6 (Loading LS12)	-251.7	194.1
LL+IM(Column)	-220.4	-452.2	343.3
LL+IM (Footing)	-185.5	N/A	301.5

Maximum Moment – Top of Pier

	Moment (K-FT/LANE)	Corresponding Axial (K/LANE)
Dual Truck Train	-769.2 (Loading 1018)	-84.6
Lane Load	-480.9 (Loading LS2)	-48.6
LL+IM (Column)	-1353.5	-145.0
LL+IM (Footing)	N/A	N/A

Appendix B**Loads and Loading****HL 93 Loading for Bridge Piers**

Code Reference

Maximum Moment – Bottom of Pier

	Moment (K-FT/LANE)	Corresponding Axial (K/LANE)
Dual Truck Train	382.7 (Loading 1018)	-84.6
Lane Load	239.1 (Loading LS2)	-48.6
LL+IM (Column)	673.3	-145.0
LL+IM (Footing)	559.6	-119.9

Maximum Shear

	Shear (K/LANE)
Dual Truck Train	28.8 (Loading 1018)
Lane Load	18.0 (Loading LS2)
LL+IM (Column)	50.7
LL+IM (Footing)	42.1

7

Transverse Analysis

Now that we have the maximum lane reactions from the longitudinal girder line analysis, we need to apply these as loads to the bent frame.

7.1 Loading

The methodology for applying superstructure live load reactions to substructure elements is described in the BDM. This methodology consists of applying the wheel line reactions directly to the crossbeam and varying the number and position of design lanes. Appendix B describes modeling techniques for GTSTRUDL™.

BDM
9.1.1.1C

7.2 Results**7.2.1 Cap Beam**

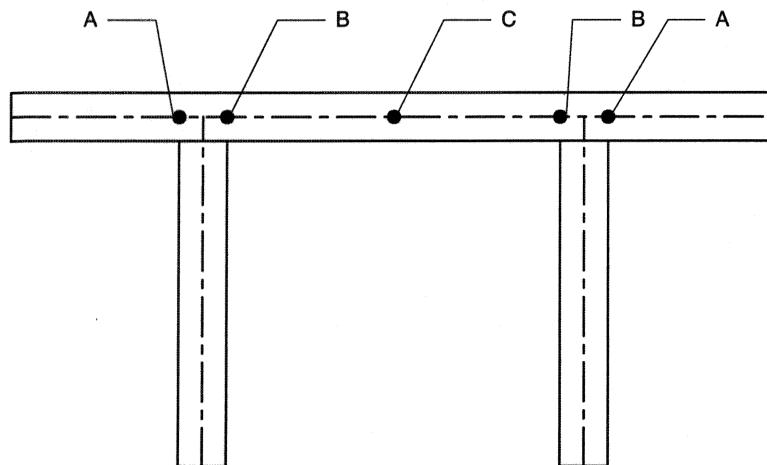
For this example, we will look at results for three design points, the left and right face of the left-hand column, and at the mid-span of the cap beam. Note that in the analysis, the wheel line reactions were applied from the left hand side of the bent. This does not result in a symmetrical set of loadings. However, because this is a symmetrical frame we expect symmetrical results. The controlling results from the left and right hand points "A" and "B" are used.

Appendix B

Loads and Loading

HL 93 Loading for Bridge Piers

Code Reference



For the shear design of the crossbeam, the LRFD specifications allow us to determine the effects of moments and shears on the capacity of the section using the maximum factored moments and shears at a section. Hence, the results below do not show the maximum shears and corresponding moments.

C5.8.3.4.2

The tables below summarize the results of the transverse analysis for the crossbeam. The basic results are adjusted with the multiple presence factors. The controlling load cases are in parentheses.

Point A

Force Effect	Shear (K) 110.2 (Loading 1009)	+Moment (K-FT) 0	-Moment (K-FT) -482.1 (1029)
Multiple Presence Factor	1.2	1.2	1.2
LL+IM	132.2	0	-578.5

Point B

	Shear (K)	+Moment (K-FT)	-Moment (K-FT)
Force Effect	155.1 (Loading 2330)	312.0 (Loading 1522)	648.0 (Loading 1029)
Multiple Presence Factor	1.0	1.2	1.2
LL+IM	155.1	374.4	-772.6

Point C

	Shear (K)	+Moment (K-FT)	-Moment (K-FT)
Force Effect	87.5 (Loading 2036)	424.6 (Loading 1520)	398.8 (Loading 1029)
Multiple Presence Factor	1.0	1.2	1.2
LL+IM	87.5	509.5	-478.6

7.2.2 Columns

The tables below show the live load results at the top and bottom of a column. The results are factored with the appropriate multiple presence factors. Controlling loads are in parentheses.

Maximum Axial

	Axial (K)	Top of Column	Bottom of Column
		Corresponding Moment (K-FT)	Corresponding Moment (K-FT)
Force Effect	-346.0 (Loading 2026)	33.9	28.2
Multiple Presence Factor	1.0	1.0	1.0
LL+IM	-346.0	33.9	28.2

Maximum Moment – Top of Column

	Moment (K-FT)	Corresponding Axial (K)
Force Effect	59.0 (Loading 1009)	-264.4
Multiple Presence Factor	1.2	1.2
LL+IM	70.8	-317.3

Maximum Moment – Bottom of Column

	Moment (K-FT)	Corresponding Axial (K)
Force Effect	-53.4 (Loading 1029)	55.4
Multiple Presence Factor	1.2	1.2
LL+IM	-64.1	66.5

Maximum Shear

	Shear (K)
Force Effect	-1.0 (Loading 1029)
Multiple Presence Factor	1.2
LL+IM	-1.2

7.2.3 Footings

Even though we didn't perform the transverse analysis with the footing loads, we can still obtain the results. Assuming we have a linear elastic system, the principle of superposition can be used. The footing results are simply the column results scaled by the ratio of the footing load to the column load. For this case, the scale factor is $185.5/220.4=0.84$.

Maximum Axial

	Axial (K)	Corresponding Moment (K-FT)
LL+IM	-290.6	23.9

Maximum Moment

	Moment (K-FT)	Corresponding Axial (K)
LL+IM	-53.8	55.8

Maximum Shear

	Shear (K)
LL+IM	-1.0

8 Combining Longitudinal and Transverse Results

To get the full set of column forces, the results from the longitudinal and transverse analyses need to be combined. Recall that the longitudinal analysis produced moments, shears, and axial load for a single loaded lane whereas the transverse analysis produced column and footing forces for multiple loaded lanes.

Before we can combine the force effects we need to determine the per column force effect from the longitudinal analysis. To do this, we look at the axial force results in transverse model to determine the lane fraction that is applied to each column.

For maximum axial load, 2 lanes at 220.4 K/LANE produce an axial force of 346 K. The lane fraction carried by the column is $346/(2 \times 220.4) = 0.785$ (78.5%).

$$M_z = (-452.2 \text{ K-FT/LANE})(2 \text{ LANES})(0.785)(1.0) = -710.0 \text{ K-FT} \text{ (Top of Column)}$$

$$M_z = (343.3 \text{ K-FT/LANE})(2 \text{ LANES})(0.785)(1.0) = 539.0 \text{ K-FT} \text{ (Bottom of Column)}$$

$$M_z = (301.5 \text{ K-FT/LANE})(2 \text{ LANES})(0.785)(1.0) = 473.4 \text{ K-FT} \text{ (Footing)}$$

For maximum moment (and shear because the same loading governs) at the top of the column, 1 lane at 220.4 K/LANE produces an axial force of 317.3. ($317.3/220.4 = 1.44$). 144% of the lane reaction is carried by the column.

$$M_z = (-1353.5)(1.44)(1.2) = -2338.8 \text{ K-FT}$$

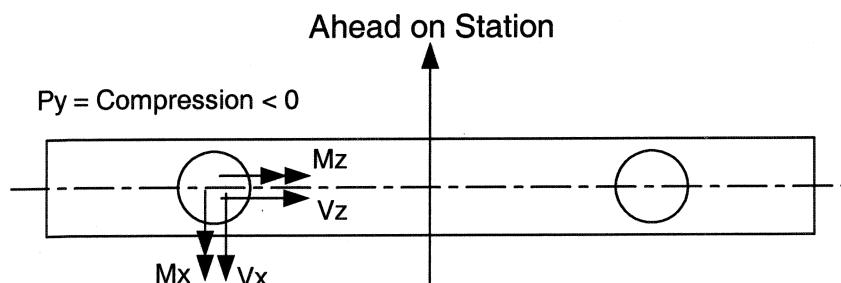
$$V_x = (50.7)(1.44)(1.2) = 87.6 \text{ K (Column)}$$

$$V_x = (42.1)(1.44)(1.2) = 72.7 \text{ K (Footing)}$$

For maximum moment at the bottom of the column, 1 lane at 220.4 K/LANE produces an axial force of 66.5 K. ($66.5/220.4 = 0.30$) 30% of the lane reaction is carried by the column.

$$M_z = (673.3)(0.30)(1.2) = 242.4 \text{ K-FT} \text{ (Column)}$$

$$M_z = (559.6)(0.30)(1.2) = 201.5 \text{ K-FT} \text{ (Footing)}$$



V_x and M_z determined from Longitudinal Analysis
P_y, V_z and M_x determined from Transverse Analysis

Appendix B**Loads and Loading****HL 93 Loading for Bridge Piers**

Code Reference

Column

	Load Case				
	Maximum Axial Top	Maximum Axial Bottom	Maximum Moment Top	Maximum Maximum Bottom	Shear
Axial (K)	-346.0	-346.0	-317.3	66.5	
Mx (K-FT)	33.6	28.2	70.8	-64.1	
Mz (K-FT)	-710.0	539.0	-2338.8	242.4	
Vx (K)					87.6
Vz (K)					-1.2

Footing

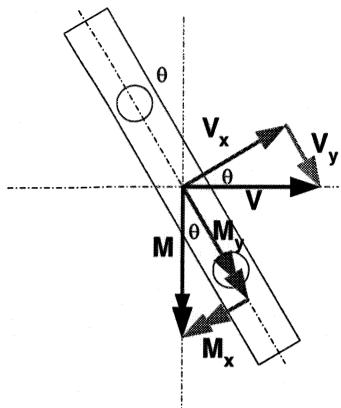
	Load Cases		
	Maximum Axial	Maximum Moment Bottom	Shear
Axial (K)	-290.6	55.8	
Mx (K-FT)	23.9	-53.8	
Mz (K-FT)	473.4	201.5	
Vx (K)			72.7
Vz (K)			-1.0

9

Skew Effects

This analysis becomes only slightly more complicated when the pier is skewed with respect to the centerline of the bridge. The results of the longitudinal analysis need to be adjusted for skew before being applied to the transverse model.

The shears and moments produced by the longitudinal analysis are in the plane of the longitudinal model. These force vectors have components that are projected into the plane of the transverse model as shown in the figure below. The transverse model loading must include these forces and moments for each wheel line load. Likewise, the skew adjusted results from the longitudinal analysis need to be used when combining results from the transverse analysis.



10 Summary

This example demonstrates a method for analyzing bridge piers subjected to the LRFD HL-93 live load. Other than the loading, the analysis procedure is the same as for the AASHTO Standard Specifications.

BRIDGE DESIGN MANUAL

Appendix B

Loads and Loading

HL 93 Loading for Bridge Piers

Code Reference

Appendix A – Longitudinal Analysis Details

This appendix shows the longitudinal analysis details. In the live load generation portion of the GTSTRUDL input, you will see multiple trials for live load analysis. Each trial uses a different range of headways pacing for the dual truck train. The first trial varies the headway spacing from 180 to 205 feet. Based on this, a tighter range between 193 and 198 feet was used to get the headway spacing corresponding to the maximum loads correct to within 1 foot.

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Fri Jun 14 12:52:45 2002

1GTICES/C-NP 2.5.0 MD-NT 2.0, January 1995.
Proprietary to Georgia Tech Research Corporation, U.S.A.

Reading password file j:\gtstrudl\gtaccess25.dat
CI-i-audfile, Command AUDIT file FILE1252.aud has been activated.

```
*** G T S T R U D L ***
RELEASE DATE      VERSION      COMPLETION NO.
August 30, 2000    25.0          4085

*** ACTIVE UNITS - LENGTH      WEIGHT      ANGLE      TEMPERATURE      TIME
*** ASSUMED TO BE   INCH       POUND      RADIAN     FAHRENHEIT     SECOND
{ 1} > $ -----
{ 2} > $ This is the Common Startup Macro; put your company-wide startup commands here.
{ 3} > $ You can edit this file from Tools -- Macros. Click "Startup" and then "Edit".
{ 4} > $ -----
{ 1} > CINPUT 'C:\Documents and Settings\briker\My -
```

BRIDGE DESIGN MANUAL

Appendix B

Loads and Loading

HL 93 Loading for Bridge Piers

Code
Reference

BRIDGE DESIGN MANUAL

Appendix B

Loads and Loading

HL 93 Loading for Bridge Piers

Code
Reference

```

{ 15) > $ ----- Boundary conditions -----
{ { 16) > 1 0.00000
{ { 17) > 2 0.00000
{ { 18) > 3 0.00000
{ { 19) > 4 0.00000
{ { 20) > 5 -40.00000 S
{ { 21) > 6 -40.00000 S
{ { 22) > $
{ { 23) > $
{ { 24) > $ -----
{ { 25) > $ --- Roller joints: rotation + horiz. translation
{ { 26) > DEFINE GROUP 'roller' ADD JOINTS 1 4
{ { 27) > STATUS SUPPORT JOINT GROUP 'roller'
{ { 28) > JOINT GRP 'roller' RELEASES FORCE X MOM Z
{ { 29) > $

{ { 30) > MEMBER INCIDENTS
{ { 31) > $ Name Start joint End joint
{ { 32) > $ -----
{ { 33) > 1 1 2
{ { 34) > 2 2 3
{ { 35) > 3 3 4
{ { 36) > 4 5 2
{ { 37) > 5 6 3
{ { 38) > $
{ { 39) > $ -----
{ { 40) > UNITS INCHES Properties -----
{ { 41) > MEMBER PROPERTIES
{ { 42) > 1 TO 3 AX 2827 IZ 636172
{ { 43) > 4 TO 5 AX 1413 IZ 318086
{ { 44) > CONSTANTS
{ { 45) > E 5538 MEMBERS 1 TO 3
{ { 46) > E 4224 MEMBERS 4 TO 5
{ { 47) > $
{ { 48) > $ -----
{ { 49) > $ ----- Loadings -----
{ { 50) > UNITS KIP FEET
{ { 51) > $
{ { 52) > $ --- Lane Loads ---
{ { 53) > LOADING 'LS12' 'Load load in span 1 and 2'
{ { 54) > MEMBER 1 2 LOAD FORCE Y UNIFORM FRACTIONAL -0.640 LA 0.0 LB 1.0
{ { 55) > 
{ { 56) > LOADING 'LS13' 'Load load in span 1 and 3'
{ { 57) > MEMBER 1 3 LOAD FORCE Y UNIFORM FRACTIONAL -0.640 LA 0.0 LB 1.0
{ { 58) > 
{ { 59) > LOADING 'LS2' 'Load load in span 2'

```

Appendix B

Loads and Loading

HL 93 Loading for Bridge Piers

	Code Reference
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```

{ 60) > MEMBER 2 LOAD FORCE Y UNIFORM FRACTIONAL -0.640 LA 0.0 LB 1.0
{ 61) > 
{ 62) > LOADING 'LS3' 'Load load in span 3'
{ 63) > MEMBER 3 LOAD FORCE Y UNIFORM FRACTIONAL -0.640 LA 0.0 LB 1.0
{ 64) > 
{ 65) > $ --- Dual Truck Train ---
{ 66) > 
{ 67) > $$ --- TRIAL 1 - (GOAL: Determine approximate headway spacing)
{ 68) > $$ --- RESULTS: Maximums occurred for headway spacings of 50' and 205'
{ 69) > $$ --- Load ID Legend
{ 70) > $$ - ID = 1000 TO 1999, 50' Headway Spacing
{ 71) > $$ - ID = 2000 TO 2999, 180' Headway Spacing
{ 72) > $$ - ID = 3000 TO 3999, 185' Headway Spacing
{ 73) > $$ - ID = 4000 TO 4999, 190' Headway Spacing
{ 74) > $$ - ID = 5000 TO 5999, 195' Headway Spacing
{ 75) > $$ - ID = 6000 TO 6999, 200' Headway Spacing
{ 76) > $$ - ID = 7000 TO 7999, 205' Headway Spacing
{ 77) > $MOVING LOAD GENERATOR
{ 78) > 
{ 79) > $SUPERSTRUCTURE FOR MEMBERS 1 TO 3
{ 80) > $STRUCK FWD GENERAL TRUCK 32.0 14.0 32.0 14.0 8.0 50.0 32.0 14.0 32.0 14.0 8.0
{ 81) > $GENERATE LOAD INITIAL 1000 PRINT OFF
{ 82) > $ 
{ 83) > $STRUCK FWD GENERAL TRUCK 32.0 14.0 32.0 14.0 8.0 180.0 32.0 14.0 32.0 14.0 8.0
{ 84) > $GENERATE LOAD INITIAL 2000 PRINT OFF
{ 85) > $ 
{ 86) > $STRUCK FWD GENERAL TRUCK 32.0 14.0 32.0 14.0 8.0 185.0 32.0 14.0 32.0 14.0 8.0
{ 87) > $GENERATE LOAD INITIAL 3000 PRINT OFF
{ 88) > $ 
{ 89) > $STRUCK FWD GENERAL TRUCK 32.0 14.0 32.0 14.0 8.0 190.0 32.0 14.0 32.0 14.0 8.0
{ 90) > $GENERATE LOAD INITIAL 4000 PRINT OFF
{ 91) > $ 
{ 92) > $STRUCK FWD GENERAL TRUCK 32.0 14.0 32.0 14.0 8.0 195.0 32.0 14.0 32.0 14.0 8.0
{ 93) > $GENERATE LOAD INITIAL 5000 PRINT OFF
{ 94) > $ 
{ 95) > $STRUCK FWD GENERAL TRUCK 32.0 14.0 32.0 14.0 8.0 200.0 32.0 14.0 32.0 14.0 8.0
{ 96) > $GENERATE LOAD INITIAL 6000 PRINT OFF
{ 97) > $ 
{ 98) > $STRUCK FWD GENERAL TRUCK 32.0 14.0 32.0 14.0 8.0 205.0 32.0 14.0 32.0 14.0 8.0
{ 99) > $GENERATE LOAD INITIAL 7000 PRINT OFF
{ 100) > $ 
{ 101) > $SEND LOAD GENERATOR
{ 102) > 
{ 103) > $ --- TRIAL 2 - (GOAL: Determine extreme values using refined headway spacing)
{ 104) > $ --- Load ID Legend

```

Appendix B

Loads and Loading

HL 93 Loading for Bridge Piers

Code
Reference

```

{ 105) > $ - ID = 1000 TO 1999, 50' Headway Spacing
{ 106) > $ - ID = 2000 TO 2999, 193' Headway Spacing
{ 107) > $ - ID = 3000 TO 3999, 194' Headway Spacing
{ 108) > $ - ID = 4000 TO 4999, 195' Headway Spacing
{ 109) > $ - ID = 5000 TO 5999, 196' Headway Spacing
{ 110) > $ - ID = 6000 TO 6999, 197' Headway Spacing
{ 111) > $ - ID = 7000 TO 7999, 198' Headway Spacing
{ 112) >
{ 113) > MOVING LOAD GENERATOR
{ 114) >

{ 115) > SUPERSTRUCTURE FOR MEMBERS 1 TO 3
{ 116) > TRUCK FWD GENERAL TRUCK 32.0 14.0 32.0 14.0 8.0 50.0 32.0 14.0 32.0 14.0 8.0
{ 117) > GENERATE LOAD INITIAL 1000 PRINT OFF
{ 118) >
{ 119) > TRUCK FWD GENERAL TRUCK 32.0 14.0 32.0 14.0 8.0 193.0 32.0 14.0 32.0 14.0 8.0
{ 120) > GENERATE LOAD INITIAL 2000 PRINT OFF
{ 121) >
{ 122) > TRUCK FWD GENERAL TRUCK 32.0 14.0 32.0 14.0 8.0 194.0 32.0 14.0 32.0 14.0 8.0
{ 123) > GENERATE LOAD INITIAL 3000 PRINT OFF
{ 124) >
{ 125) > TRUCK FWD GENERAL TRUCK 32.0 14.0 32.0 14.0 8.0 195.0 32.0 14.0 32.0 14.0 8.0
{ 126) > GENERATE LOAD INITIAL 4000 PRINT OFF
{ 127) >
{ 128) > TRUCK FWD GENERAL TRUCK 32.0 14.0 32.0 14.0 8.0 196.0 32.0 14.0 32.0 14.0 8.0
{ 129) > GENERATE LOAD INITIAL 5000 PRINT OFF
{ 130) >
{ 131) > TRUCK FWD GENERAL TRUCK 32.0 14.0 32.0 14.0 8.0 197.0 32.0 14.0 32.0 14.0 8.0
{ 132) > GENERATE LOAD INITIAL 6000 PRINT OFF
{ 133) >
{ 134) > TRUCK FWD GENERAL TRUCK 32.0 14.0 32.0 14.0 8.0 198.0 32.0 14.0 32.0 14.0 8.0
{ 135) > GENERATE LOAD INITIAL 7000 PRINT OFF
{ 136) >
{ 137) > END LOAD GENERATOR
** OUT OF MOVING LOAD GENERATOR
{ 138) > $ ----- Analysis
{ 139) > $ -----
{ 140) > $ -----
{ 141) > STIFFNESS ANALYSIS
TIME FOR CONSISTENCY CHECKS FOR 5 MEMBERS 0.00 SECONDS
TIME FOR BANDWIDTH REDUCTION 0.00 SECONDS
TIME TO GENERATE 5 ELEMENT STIF. MATRICES 0.00 SECONDS
TIME TO PROCESS 1337 MEMBER LOADS 0.02 SECONDS
TIME TO ASSEMBLE THE STIFFNESS MATRIX 0.00 SECONDS
TIME TO PROCESS 6 JOINTS 0.01 SECONDS
TIME TO SOLVE WITH 1 PARTITIONS 0.00 SECONDS

```

BRIDGE DESIGN MANUAL

Appendix B

Loads and Loading

HL 93 Loading for Bridge Piers

**Code
Reference**

```

TIME TO PROCESS      6 JOINT DISPLACEMENTS    0.02 SECONDS
TIME TO PROCESS      5 ELEMENT DISTORTIONS   0.03 SECONDS
TIME FOR STATICS CHECK          0.00 SECONDS

{ 142} > $ -----
{ 143} > $ ----- Results
{ 144} > $ -----
{ 145} > OUTPUT BY MEMBER
{ 146} > $ -----
{ 147} > $ ----- Dual Truck Results Envelope (top and bottom of pier)
{ 148} > LOAD LIST 1000 TO 7999
{ 149} > LIST FORCE ENVELOPE MEMBER 4 SECTION FRACTIONAL NS 2 1.0 0.0
{ 150} >

```

*****RESULTS OF LATEST ANALYSES*****

PROBLEM - NONE	TITLE - NONE GIVEN				
ACTIVE UNITS	FEET KIP RAD DEGF SEC				

INTERNAL MEMBER RESULTS

MEMBER FOBCE ENVELOPE

BRIDGE DESIGN MANUAL

Appendix B

Loads and Loading

HL 93 Loading for Bridge Piers

**Code
Reference**

```

1 { 150) > $ ----- Lane Load Results Envelope (top and bottom of pier)
1 { 151) > LOAD LIST 'LS12' 'LS13' 'LS2' 'LS3'
1 { 152) > LIST FORCE ENVELOPE MEMBER 4 SECTION FRACTIONAL NS 2 1.0 0.0
1 { 153) >
1
*****
*RESULTS OF LATEST ANALYSES*
*****

```

PROBLEM - NONE TITLE - NONE GIVEN

ACTIVE UNITS FEET KTP BAD DECE SEC

INTERNAL MEMBER RESULTS

MEMBER FORCE ENVIRONMENT

MEMBER 4

DISTANCE FROM START	/	FORCE AXIAL	Y SHEAR	Z SHEAR	/	TORSION	Y BENDING	MOMENT Z BENDING	/
1.000 FR		2.951640	18.00047					366.3766	
		LS3		LS2				LS13	
		-88.62376	-13.71255					-480.9429	
		LS12		LS13				LS2	
0.000		2.951640	18.00047					239.0760	
		LS3		LS2				LS2	
		-88.62376	-13.71255					-182.1253	
		LS12		LS13				LS13	

BRIDGE DESIGN MANUAL

Appendix B

Loads and Loading

HL 93 Loading for Bridge Piers

Code
Reference

```
{ 157} > LIST SECTION FORCES MEMBER 4 SECTION FRACTIONAL NS 2 1.0 0.0
```

```
*****  
*RESULTS OF LATEST ANALYSES*  
*****
```

```
PROBLEM - NONE      TITLE - NONE GIVEN
```

```
ACTIVE UNITS   FEET KIP RAD DEGF SEC
```

```
INTERNAL MEMBER RESULTS
```

```
MEMBER SECTION FORCES
```

```
-----  
-- MEMBER 4  
-----
```

```
LOADING   LS12      Load load in span 1 and 2
```

DISTANCE FROM START		FORCE	Z SHEAR	TORSION	MOMENT
		AXIAL	Y SHEAR		Z BENDING
1.000 FR	-88	62377	11.14420		-251.7068
0.000	-88	62377	11.14420		194.0612

```
LOADING   LS2      Load load in span 2
```

DISTANCE FROM START		FORCE	Z SHEAR	TORSION	MOMENT
		AXIAL	Y SHEAR		Z BENDING
1.000 FR	-48	61551	18.00047		-480.9430
0.000	-48	61551	18.00047		239.0760

```
LOADING   1014      USERS TRUCK FORWARD PIVOT ON SECTION 2 MEMBER 2
```

DISTANCE		FORCE			MOMENT
					/

BRIDGE DESIGN MANUAL**Appendix B****Loads and Loading****HL 93 Loading for Bridge Piers**Code
Reference

FROM START	AXIAL	Y SHEAR	Z SHEAR	TORSION	Y BENDING	Z BENDING
1.000 FR	-117.5348	8.235572				-188.5018
0.000	-117.5348	8.235572				140.9211
LOADING	1018	USERS TRUCK	FORWARD	PIVOT ON SECTION 6	MEMBER 2	
DISTANCE FROM START	/-----/	AXIAL	FORCE	/-----/	MOMENT	/-----/
1.000 FR	-84.59298	28.79776	Y SHEAR	Z SHEAR	Y BENDING	Z BENDING
0.000	-84.59298	28.79776				

Appendix B – Transverse Analysis Details

This appendix shows the details of the transverse analysis. The interesting thing to note about the transverse analysis is the live load truck configuration. A technique of treating the wheel line reactions as a longitudinal live load is used. A two axle “truck” is created. The truck is positioned so that it is on the left edge, center, and right edge of the design lane. In order to keep the axles in the correct position, a dummy axle with a weight of 0.0001 kips was used. This dummy axial is the lead axle of the truck and it is positioned in such a way as to cause the two “real” axles to fall in the correct locations within the design lanes.

The GTSTRUDL live load generator uses partial trucks when it is bring a truck onto or taking it off a bridge. As such, less then the full number of axles are applied to the model. For the transverse analysis, we do not want to consider the situation when only one of the two wheel lines is on the model. As such, several load cases are ignored by way of the LOAD LIST command on line 76 of the output.

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Fri Jun 14 13:13:09 2002

1GTICES/C-NP 2.5.0 MD-NT 2.0, January 1995.
Proprietary to Georgia Tech Research Corporation, U.S.A.

Reading password file j:\gtstrudl\gtaccess25.dat
CI-i-audfile, Command AUDIT file FILE1313.aud has been activated.

*** G T S T R U D L ***	VERSION	COMPLETION NO.
RELEASE DATE	25.0	4085
August 30, 2000		
***** ACTIVE UNITS -	LENGTH	ANGLE
***** ASSUMED TO BE	INCH	POUND
		RADIAN
		TEMPERATURE
		FAHRENHEIT
		TIME
		SECOND

```
{
  1} > $ -----
{ 2} > $ This is the Common Startup Macro; put your company-wide startup commands
here.
{ 3} > $ You can edit this file from Tools -- Macros. Click "Startup" and then
"Edit".
{ 4} > $ -----
```

BRIDGE DESIGN MANUAL

Appendix B

Loads and Loading

HL 93 Loading for Bridge Piers

Code Reference

```
{ 1} > C\INPUT 'C:\Documents and Settings\bricer\My -  
{ 2} > _Documents\LiveLoad\Transverse.gti'  
{ 3} > $ -----  
{ 4} > $ Live Load Pier Analysis Example  
{ 5} > $ Transverse Analysis to determine column loads  
{ 6} > $ -----  
{ 7} > $  
{ 8} > STRUDL
```

	ACTIVE UNITS - ASSUMED TO BE	LENGTH INCH	WEIGHT POUND	ANGLE RADIAN	TEMPERATURE FAHRENHEIT	TIME SECOND
{ 9 } > TYPE PLANE FRAME XY	{ 10 } > MATERIAL STEEL	{ 11 } > OUTPUT LONG NAME	{ 12 } > UNITS FEET KIPS			

BRIDGE DESIGN MANUAL

Appendix B

Loads and Loading

HL 93 Loading for Bridge Piers

Code
Reference

```

{ 13) > $ JOINT COORDINATES
{ { 14) > $ Name X coord Y coord
{ { { 15) > $ ----- -----
{ { { 16) > $ ----- -----
{ { { 17) > $ ----- -----
{ { { 18) > 1 -14.00000 40.00000
{ { { 19) > 2 -7.00000 40.00000
{ { { 20) > 3 7.00000 40.00000
{ { { 21) > 4 14.00000 40.00000
{ { { 22) > 5 -7.00000 0.00000 S
{ { { 23) > 6 7.00000 0.00000 S
{ { { 24) > $ -----
{ { { 25) > MEMBER INCIDENCES
{ { { 26) > $ Name Start joint End joint
{ { { { 27) > $ ----- -----
{ { { { 28) > 1 1 2
{ { { { 29) > 2 2 3
{ { { { 30) > 3 3 4
{ { { { 31) > 4 5 2
{ { { { 32) > 5 6 3
{ { { { 33) > $ ----- -----
{ { { { 34) > $ ----- -----
{ { { { 35) > UNITS INCHES
{ { { { 36) > MEMBER PROPERTIES
{ { { { 37) > 1 TO 3 AX 64935 IZ 6283008 $ CAP BEAM
{ { { { 38) > 4 TO 5 AX 2827 IZ 636172 $ COLUMNS
{ { { { 39) > UNITS FEET
{ { { { 40) > $ ----- -----
{ { { { 41) > $ ----- -----
{ { { { 42) > $ ----- -----
{ { { { 43) > MOVING LOAD GENERATOR
{ { { { 44) > SUPERSTRUCTURE FOR MEMBERS 1 TO 3
{ { { { 45) > $ ----- -----
{ { { { 46) > $ One lane loaded - Left Aligned
{ { { { 47) > TRUCK FWD GENERAL TRUCK NP 3 110.2 6 110.2 0.875 0.0001
{ { { { 48) > GENERATE LOAD INITIAL 1000 PRINT OFF
{ { { { 49) > $ One lane loaded - Center Aligned
{ { { { 50) > TRUCK FWD GENERAL TRUCK NP 3 110.2 6 110.2 2.125 0.00001
{ { { { 51) > GENERATE LOAD INITIAL 1300 PRINT OFF
{ { { { 52) > $ ----- -----
{ { { { 53) > $ One lane loaded - Right Aligned
{ { { { 54) > TRUCK FWD GENERAL TRUCK NP 3 110.2 6 110.2 3.125 0.0001
{ { { { 55) > GENERATE LOAD INITIAL 1500 PRINT OFF
{ { { { 56) > $ ----- -----
{ { { { 57) >

```

BRIDGE DESIGN MANUAL

Appendix B

Loads and Loading

HL 93 Loading for Bridge Piers

Code
Reference

```

{
  { 13} > $ 
  { 14} > JOINT COORDINATES
  { 15} > $ Name ----- X coord ----- Y coord
  { 16} > $ ----- ----- ----- ----- 
  { 17} > $ 1 -14.00000 40.00000
  { 18} > $ 2 -7.00000 40.00000
  { 19} > $ 3 7.00000 40.00000
  { 20} > $ 4 14.00000 40.00000
  { 21} > $ 5 -7.00000 0.00000 S
  { 22} > $ 6 0.00000 S
  { 23} > $ 
  { 24} > $ 
  { 25} > MEMBER INCIDENCES
  { 26} > $ Name Start joint End joint
  { 27} > $ ----- ----- ----- ----- 
  { 28} > $ 1 1 1 2
  { 29} > $ 2 2 3
  { 30} > $ 3 3 4
  { 31} > $ 4 5 2
  { 32} > $ 5 6 3
  { 33} > $ 
  { 34} > $ ----- Properties ----- 
  { 35} > UNITS INCHES
  { 36} > MEMBER PROPERTIES
  { 37} > 1 TO 3 AX 64935 IZ 6283008 $ CAP BEAM
  { 38} > 4 TO 5 AX 2827 IZ 636172 $ COLUMNS
  { 39} > UNITS FEET
  { 40} > $ 
  { 41} > $ ----- Loadings ----- 
  { 42} > $ 
  { 43} > MOVING LOAD GENERATOR
  { 44} > SUPERSTRUCTURE FOR MEMBERS 1 TO 3
  { 45} > $ 
  { 46} > $ One lane loaded - Left Aligned
  { 47} > TRUCK FWD GENERAL TRUCK NP 3 110.2 6 110.2 0.875 0.0001
  { 48} > GENERATE LOAD INITIAL 1300 PRINT OFF
  { 49} > $ 
  { 50} > $ One lane loaded - Center Aligned
  { 51} > TRUCK FWD GENERAL TRUCK NP 3 110.2 6 110.2 2.125 0.0001
  { 52} > GENERATE LOAD INITIAL 1300 PRINT OFF
  { 53} > $ 
  { 54} > $ One lane loaded - Right Aligned
  { 55} > TRUCK FWD GENERAL TRUCK NP 3 110.2 6 110.2 3.125 0.0001
  { 56} > GENERATE LOAD INITIAL 1500 PRINT OFF
  { 57} > $ 
}

```

BRIDGE DESIGN MANUAL

Appendix B

Loads and Loading

HL 93 Loading for Bridge Piers

Code
Reference

```
{ { 58} > $ Two lanes loaded - Left Aligned
{ { 60} > TRUCK FWD GENERAL TRUCK NP 5 110.2 6 110.2 6 110.2 0.875 0.0001
{ { 61} > GENERATE LOAD INITIAL 2000 PRINT OFF
{ { 62} >
{ { 63} > $ Two lanes loaded - Center Aligned
{ { 64} > TRUCK FWD GENERAL TRUCK NP 5 110.2 6 110.2 6 110.2 2.125 0.00001
{ { 65} > GENERATE LOAD INITIAL 2300 PRINT OFF
{ { 66} >
{ { 67} > $ Two lanes loaded - Right Aligned
{ { 68} > TRUCK FWD GENERAL TRUCK NP 5 110.2 6 110.2 6 110.2 3.125 0.0001
{ { 69} > GENERATE LOAD INITIAL 2500 PRINT OFF
{ { 70} >
{ { 71} > END LOAD GENERATOR
{ { 72} > $ -----
{ { 73} > $ ----- Analysis
{ { 74} > $ -----
{ { 75} > $ --- Keep active only those loads where all of the "axles" are on the structure
{ { 76} > LOAD LIST 1009 TO 1029 1311 TO 1330 1513 TO 1531 2026 TO 2037 2328 TO 2338 2530 TO 2539
{ { 77} > STIFFNESS ANALYSIS
TIME FOR CONSISTENCY CHECKS FOR 5 MEMBERS 0.01 SECONDS
TIME FOR BANDWIDTH REDUCTION 0.00 SECONDS
TIME TO GENERATE 5 ELEMENT STIFF. MATRICES 0.00 SECONDS
TIME TO PROCESS 345 MEMBER LOADS 0.00 SECONDS
TIME TO ASSEMBLE THE STIFFNESS MATRIX 0.00 SECONDS
TIME TO PROCESS 6 JOINTS 0.00 SECONDS
TIME TO SOLVE WITH 1 PARTITIONS 0.00 SECONDS
TIME TO PROCESS 6 JOINT DISPLACEMENTS 0.01 SECONDS
TIME TO PROCESS 5 ELEMENT DISTORTIONS 0.00 SECONDS
TIME FOR STATICS CHECK 0.00 SECONDS
{ { 78} > $ -----
{ { 79} > $ ----- Results
{ { 80} > $ -----
{ { 81} > $ CAP BEAM RESULTS (FACE OF COLUMN AND CENTERLINE BEAM)
{ { 82} > LIST FORCE ENVELOPE MEMBER 1 SECTION NS 1 4 . 5
```

RESULTS OF LATEST ANALYSES

PROBLEM - NONE TITLE - NONE GIVEN

BRIDGE DESIGN MANUAL

Appendix B

Loads and Loading

HL 93 Loading for Bridge Piers

Code
Reference

ACTIVE UNITS FEET KIP RAD DEGF SEC

INTERNAL MEMBER RESULTS

MEMBER FORCE ENVELOPE

--- MEMBER 1 ---

DISTANCE FROM START	FORCE			MOMENT		
	AXIAL	Y SHEAR	Z SHEAR	TORSION	Y BENDING	Z BENDING
4.500	0.000000E+00 1009	110.2001 1009			0.4843501E-11 2539	
	0.000000E+00	-0.3330342E-11 2034			-399.4755 1009	

1 { 83} > LIST FORCE ENVELOPE MEMBER 2 SECTION NS 3 2.5 7 11.5
1

RESULTS OF LATEST ANALYSES

PROBLEM - NONE TITLE - NONE GIVEN

ACTIVE UNITS FEET KIP RAD DEGF SEC

INTERNAL MEMBER RESULTS

MEMBER FORCE ENVELOPE

--- MEMBER 2 ---

BRIDGE DESIGN MANUAL

Appendix B

Loads and Loading

HL 93 Loading for Bridge Piers

Code
Reference

DISTANCE FROM START	/ ----- AXIAL	FORCE Y SHEAR	-----//----- Z SHEAR	TORSION	Y BENDING	-----//----- Z BENDING
2.500	1.059774 1029 -0.7793369 1021	55.39512 1029 -155.1088 2330				312.9793 1522 -519.6652 1009
7.000	1.059774 1029 -0.7793369 1021	87.52518 2036 -87.52517 2328				424.5728 1520 -398.638 1029
11.500	1.059774 1029 -0.7793369 1021	155.1088 2034 -44.01807 1009				299.8212 1022 -648.0418 1029

1 { 84} > LIST FORCE ENVELOPE MEMBER 3 SECTION NS 1 2.5

RESULTS OF LATEST ANALYSES

PROBLEM - NONE TITLE - NONE GIVEN

ACTIVE UNITS FEET KIP RAD DEGFF SEC

INTERNAL MEMBER RESULTS

MEMBER FORCE ENVELOPE

DISTANCE FROM START	/ ----- AXIAL	FORCE Y SHEAR	-----//----- Z SHEAR	TORSION	Y BENDING	-----//----- Z BENDING
	----- MEMBER 3					

BRIDGE DESIGN MANUAL

Appendix B

Loads and Loading

HL 93 Loading for Bridge Piers

Code
Reference

```
2.500      0.194455E-10   0.1487361E-11    0.7006327E-05
           1529          2328          2533
           0.000000E+00   -110.2000     -482.1250
           1010          1026          1029
```

```
{ 85) > $ COLUMN TOP AND BOTTOM RESULTS
{ 86) > LIST FORCE ENVELOPE MEMBER 4 SECTION FRACTIONAL NS 2 1.0 0.0
{ 87) > INTERNAL MEMBER RESULTS
}
*****RESULTS OF LATEST ANALYSES*****
*****RESULTS OF LATEST ANALYSES*****
*****RESULTS OF LATEST ANALYSES*****
```

PROBLEM - NONE TITLE - NONE GIVEN

ACTIVE UNITS FEET KIP RAD DEGF SEC

INTERNAL MEMBER RESULTS

MEMBER FORCE ENVELOPE

----- MEMBER 4 -----

DISTANCE FROM START	/----- AXIAL	FORCE			/-----// Z SHEAR	TORSION	Y BENDING	MOMENT Z BENDING
		X	Y	Z				
1.000 FR	55.39512 0.7793369	1029 1021	-345.9757 -1.05974	2026 1029				59.04022 1009
0.000	55.39512 0.7793369	1029 1021	-345.9757 -1.05974	2026 1029				-26.88208 2539
								28.22849 2026
								-53.38884 1029

Appendix B

Loads and Loading

HL 93 Loading for Bridge Piers

Code Reference

```

{ 88} > $ RESULTS CORRESPONDING TO MIN/MAX VALUES
{ 89} > $ Corresponding values not needed for cross beam
{ 90} > $ COLUMN TOP AND BOTTOM RESULTS
{ 91} > $ LOAD LIST 1009 1029 2026 2539
{ 92} > LIST SECTION FORCES MEMBER 4 SECTION FRACTIONAL NS 2 1.0 0.0
{ 93} >

```

* RESULTS OF LATEST ANALYSES *

PROBLEM - NONE TITLE - NONE GIVEN

ACTIVE UNITS FEET KIP BAD DEGE SEC

INTERNATIONAL MEMBER RESULTS

MEMBER SECTION FORCES

LOADING	1009	USERS TRUCK	FORWARD	PIVOT ON SECTION 0	MEMBER 1	
DISTANCE FROM START	/-----\	AXIAL	FORCE Y SHEAR	/-----\	TORSION	MOMENT Y BENDING
1.000 FR	-264.4182 -264.4182		-0.8546570 -0.8546570			59.04022 24.85394
LOADING	1029	USERS TRUCK	FORWARD	PIVOT ON SECTION 0	MEMBER 3	
DISTANCE FROM START	/-----\	AXIAL	FORCE Y SHEAR	/-----\	TORSION	MOMENT Y BENDING
1.000 FR	55.39512 55.39512 0.000		-1.059774 -1.059774 0.000			-10.99789 -53.38884

BRIDGE DESIGN MANUAL**Appendix B****Loads and Loading****HL 93 Loading for Bridge Piers**Code
Reference

LOADING	2026	USERS TRUCK	FORWARD	PIVOT ON SECTION 0	MEMBER 1	MOMENT	Z BENDING
<hr/>							
DISTANCE FROM START	/	AXIAL	FORCE	/	/	/	/
		Y	Y SHEAR	Z	TORSION	Y	BENDING
1.000 FR	-345.9757	-0.1419347					
0.000	-345.9757	-0.1419347					
<hr/>							
LOADING	2539	USERS TRUCK	FORWARD	PIVOT ON SECTION 9	MEMBER 1	MOMENT	Z BENDING
DISTANCE FROM START	/	AXIAL	FORCE	/	/	/	/
		Y	Y SHEAR	Z	TORSION	Y	BENDING
1.000 FR	-85.69238	-0.2037623					
0.000	-85.69238	-0.2037623					

BRIDGE DESIGN MANUAL

Appendix B

Loads and Loading

Seismic Analysis Example

A recent analysis of a bridge on I-90 in the Mercer Slough area near Bellevue provides the following example:

The deep soft soil at the site is classified as "Type III" from the AASHTO Specifications. An acceleration coefficient of 0.25, see Figure 4.1.5-1, was selected as appropriate.

The acceleration spectrum shown in Appendix 4.3-B3-2 was used to load the bridge. The results which SEISAB calculated for the first 6 modes of oscillation appear in Appendix 4.3-B3-3. The CS values in the table relate directly to the response periods of the various modes as solutions to the equation:

$$C_s = \frac{1.2AS}{T^{2/3}}$$

where:

A = The acceleration coefficient

S = The soil profile coefficient (1.5 in this case)

T = The period of vibration of the bridge, the time it takes for one cycle of oscillation

In an undamped, single degree of freedom system, the natural period is defined as:

$$T = \pi \sqrt{\frac{M}{K}}$$

where:

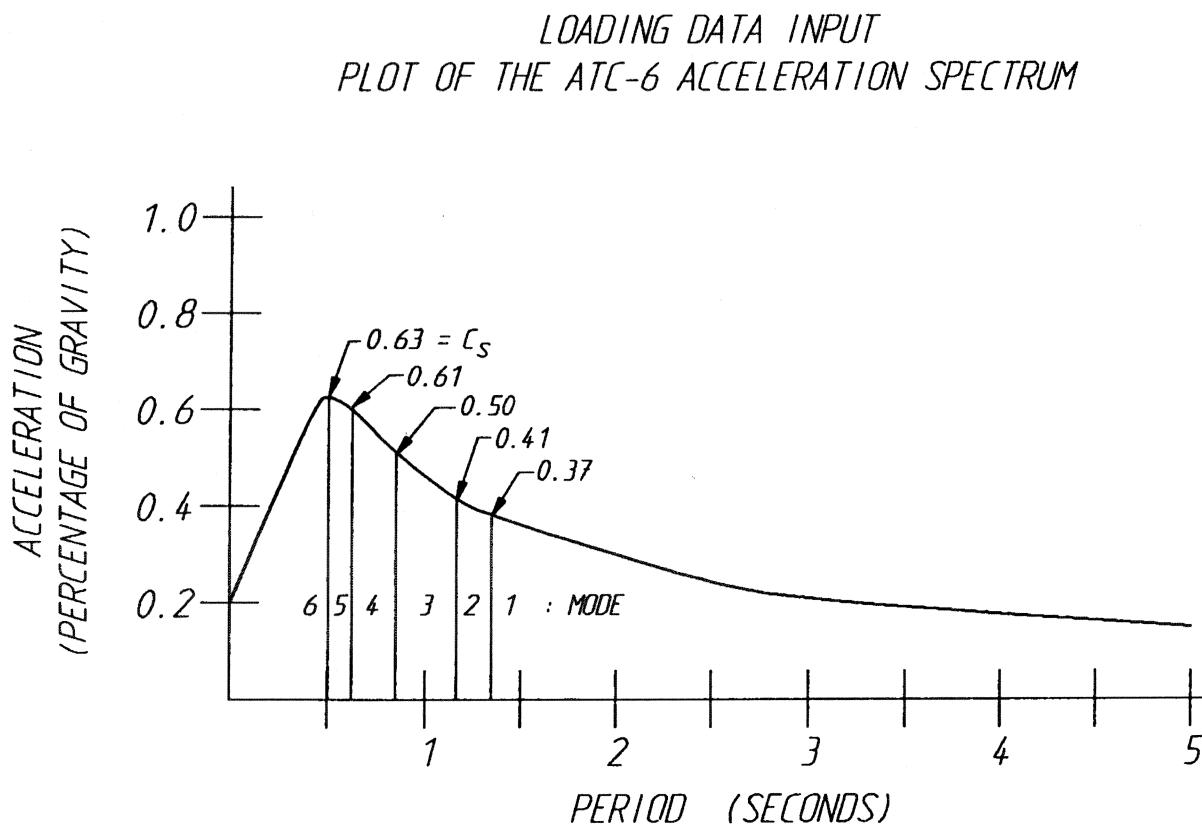
M = The mass involved

K = The spring constant

See Bibliography 1 and 7 for further comments and procedures.

CS, the elastic seismic response coefficient, is the percentage of a gravity force which is applied to the bridge for a particular mode. The participation factors indicate that modes 1 and 3 contribute most heavily to the design forces. In this case, the ground sends 0.25 g and the bridge receives about 0.50 g.

The 0.50 g applied, divided by R = 5, translates to 0.1 g when figuring design moments for a multiple column bent. Design shears would be the lesser of the values produced by 0.50 g and the shears associated with plastic hinging moments. Since the column reinforcement may yield when the 0.1 g level is reached, the energy remaining will be redistributed to the remainder of the bridge. The main column reinforcement must be adequately confined by ties or spirals to allow redistribution to occur while maintaining structural integrity.

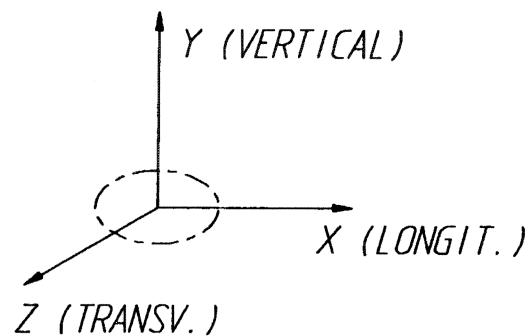


Example Seismic Analysis

RESPONSE SPECTRUM RESULTS

VIBRATIONS CHARACTERISTICS

MODE	PERIOD	FREQUENCY	CS	PARTICIPATION FACTORS		
				X	Y	Z
1	1.346	0.743	0.37	0.736	0.001	6.941 PRIMARY TRANS.
2	1.165	0.858	0.41	-0.191	0.001	-0.863
3	0.850	1.177	0.50	-6.996	-0.362	0.757 PRIMARY LONGIT.
4	0.628	1.593	0.61	-1.064	1.439	0.114
5	0.506	1.978	0.63	-0.695	0.779	0.369
6	0.497	2.012	0.63	-0.314	0.180	-1.362

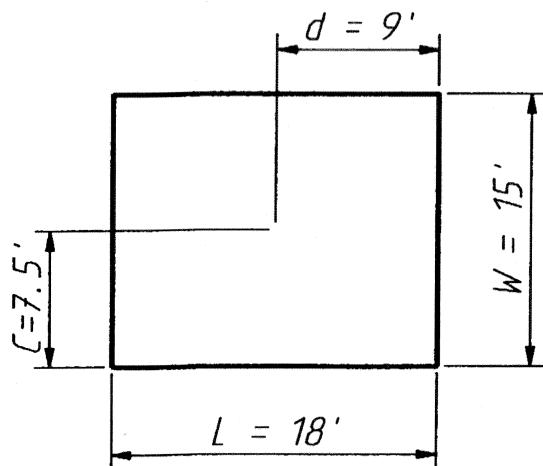


Example Seismic Analysis (Continued)

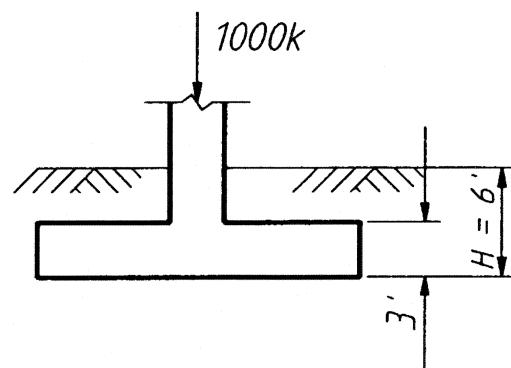
Appendix B

Loads and Loading

Spring Constants Evaluation Example



PLAN



ELEVATION

Given Data

- Cohesionless soil – Poisson's ratio = 0.33 = μ
- Soil density – 120pcf = σ
- V_s = shear wave velocity = 1,500 ft/sec

Solution:

Shear Modulus

$$G = \sigma V_s^2 = \frac{120 \text{ lb/ft}^3 (1,500 \text{ ft/sec})^2}{32.2 \text{ ft/sec}^2 (1000 \text{ lb/in}^2)}$$

Vertical Stiffness

L/W;	1.0	1.5	2.0	3.0	5.0	10.0
β_z ;	2.12	2.14	2.18	2.26	2.44	2.82

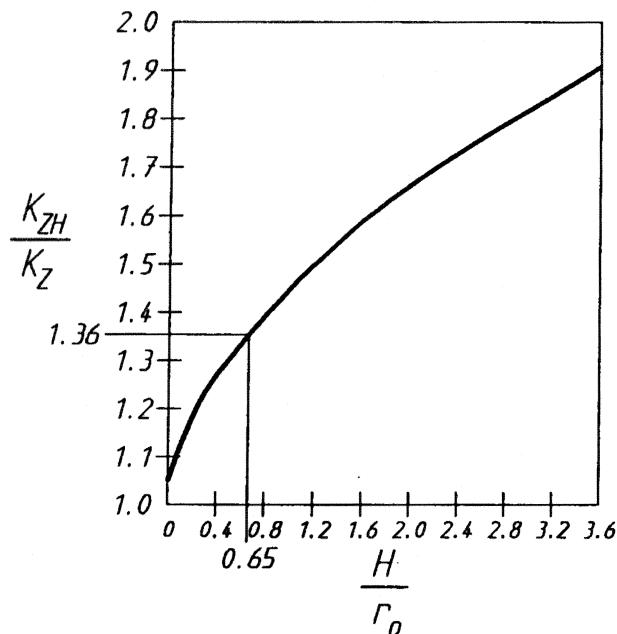
$$\frac{L}{W} = \frac{18}{15} = 1.20 \quad \beta_z = 2.13$$

$$K_z = \frac{\beta_z G \sqrt{LW}}{1-\mu} = \frac{2.13 \times 8385 \sqrt{18 \times 15}}{1-0.33} = 438,000 \frac{\text{K}}{\text{ft}}$$

Embedment Factor

$$r_o = \sqrt{\frac{Kw}{\pi}} = 9.27'$$

$$\frac{H}{r_o} = \frac{6}{9.27} = 0.65$$

Appendix B**Loads and Loading****Spring Constants Evaluation Example**

Vertical Stiffness — Modified

$$K_{ZH} = 1.36 K_Z = 1.36 \times 438,000 = 596,000 \text{ kips/ft} = KFY$$

Horizontal Stiffness

$$\frac{L}{W} = 1.20 < 5 \quad \beta_x = 2.0 \quad (\text{See page 6-37 of Bibliography 2 for explanation.})$$

$$\begin{aligned} K_x &= \beta_x (1 - \mu) \quad G \sqrt{LW} \\ &= 2.0 (1 - 0.33) \quad 8385 \sqrt{18 \times 15} = 185,000 \text{ K/ft} \end{aligned}$$

Assuming that the horizontal embedment effect is the same as the vertical.

Horizontal Stiffness — Modified

$$K_{xH} = 1.85 \times 10^5 \quad 1.36 = 2.5 \times 10^5 \text{ K/ft} = KFX = KFZ$$

Appendix B

Loads and Loading

Spring Constants Evaluation Example

Rocking Stiffness

Long Direction $c = 7.5'$ $d = 9'$

$$R = \frac{d}{c} = 1.20 \quad \beta\psi = 0.52$$

R;	0.2	0.5	1.0	2.0	4.0	6.0	8.0
$\beta\psi$;	0.4	0.45	0.5	0.6	0.8	0.95	1.1

$$K\psi = \beta\psi \frac{(8Gcd^2)}{1-\mu}$$

$$= \frac{0.52 \times 8 \times 8385 \times 7.5 \times 9^2}{1 - 0.33} = 3.2 \times 10^7 \frac{\text{K-ft}}{\text{rad}}$$

$$K_H = 1.36 (3.2 \times 10^7) = 4.3 \times 10^7 \frac{\text{K-ft}}{\text{rad}} = \text{KMZ}$$

Short Direction

$$R = \frac{c}{d} = 0.83 \quad \beta\psi = 0.48$$

$$K\psi = \beta\psi \frac{(8G)dc^2}{1-\mu} = 2.4 \times 10^7 \frac{\text{K-ft}}{\text{rad}}$$

$$= \frac{0.48 \times 8 \times 8385 \times 9 \times 7.5^2}{1 - 0.33}$$

$$K\psi H = 1.36 (2.4 \times 10^7) = 3.3 \times 10^7 \frac{\text{K-ft}}{\text{rad}}$$

Torsional Stiffness

$$r_e = \sqrt[4]{\frac{16cd(c^2 + d^2)}{6\pi}} = \sqrt[4]{\frac{16 \times 7.5 \times 9(7.5^2 + 9^2)}{6\pi}}$$

$$K_\theta = \frac{16}{3} G r_e^3 = \frac{16}{3} \times 8385 \times 9.42^3 = 3.7 \times 10^7 \frac{\text{K-ft}}{\text{rad}}$$

$$K_\theta H = 1.36 (3.7 \times 10^7) = 5.0 \times 10^7 \frac{\text{K-ft}}{\text{rad}} = \text{KMY}$$

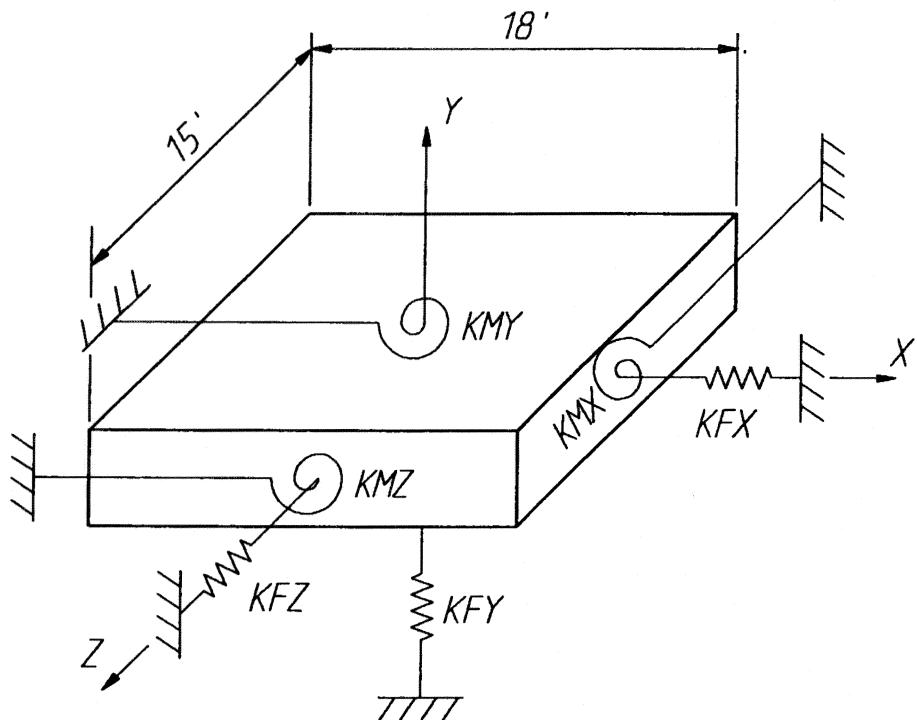
Appendix 4.4-B1-4 depicts the footing from the example in spring matrix form. The nomenclature is general, and is used for GTSTRUDL input (GTSTRUDL 4.2.2d contains a similar matrix using SEISAB nomenclature).

BRIDGE DESIGN MANUAL

Appendix B

Loads and Loading

Spring Constants Evaluation Example



KFX	KFY	KFZ	KMX	KMY	KMZ
2.5×10^5	0	0	0	0	0
0	6.0×10^5	0	0	0	0
0	0	2.5×10^5	0	0	0
0	0	0	3.3×10^7	0	0
0	0	0	0	5.0×10^7	0
0	0	0	0	0	4.3×10^7

Spring Matrix